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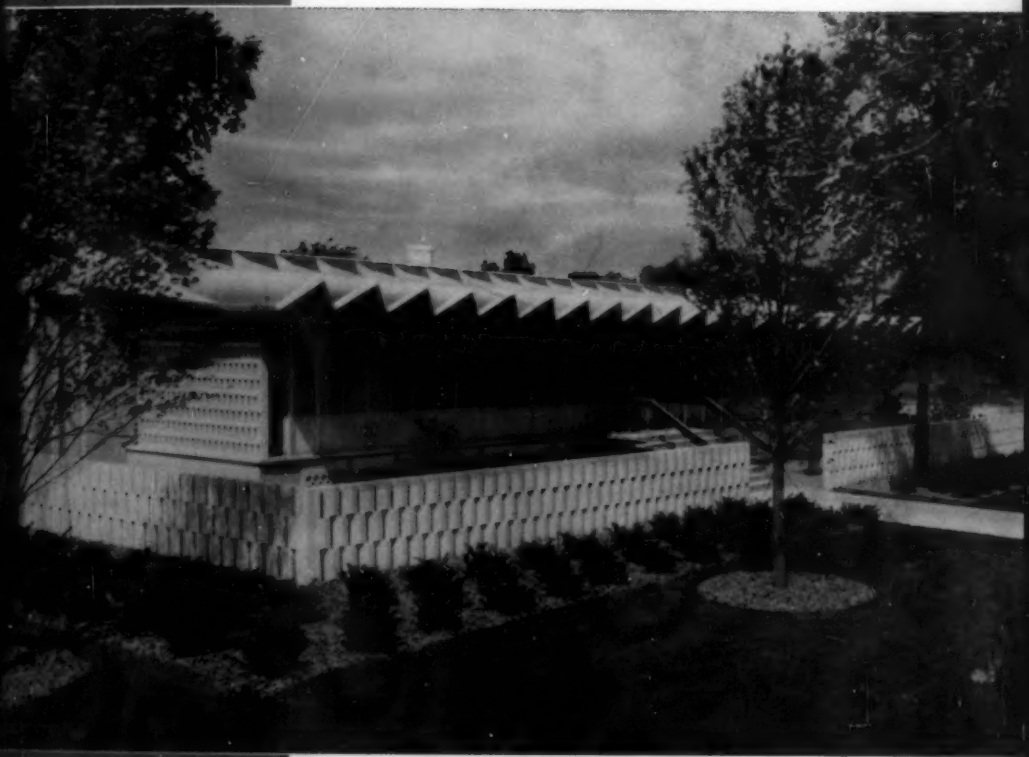

ACI
PROCEEDINGS
VOL. 55

DEDICATION ISSUE . . . Commemorating the official dedication of the American Concrete Institute headquarters building in Detroit, October 29, 1958

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OCTOBER 1958

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JOURNAL
OF THE
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the ACI JOURNAL

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Coming next month in the JOURNAL

"Proposed ACI Standard Recommended Practice for Hot Weather Concreting" prepared by ACI COMMITTEE 605, lead article in the November JOURNAL, will be considered for adoption by the Institute at the Los Angeles convention next February. ACI COMMITTEE 614 presents "Proposed Revision of Recommended Practice for Measuring, Mixing, and Placing Concrete," also intended for consideration at Los Angeles to supersede ACI 614-42.

●

"Load Factors" reported by the INTERNATIONAL COUNCIL FOR BUILDING RESEARCH offers recommendations by an eminent European committee following their study of various sources of error in assumptions, design, workmanship, and evaluation of materials. The report was accepted by the European Committee on Concrete in Rome, April, 1957.

GEORGE C. ERNST provides a highly informative report on the results of an investigation on "Tests on Moment and Shear Redistribution in Two-Span Continuous Reinforced Concrete Beams" under extreme conditions of support displacement.

●

"Influence of Water-Cement Ratio on the Behavior of a Cement-Sand Mortar in which Shrinkage Is Partially Restrained," by F. A. BLAKEY, studies stresses which actually cause cracking under slow loading imposed by restrained shrinkage.

Coauthors JACK R. BENJAMIN and HARRY A. WILLIAMS report their observations on the "Behavior of One-Story Reinforced Concrete Walls Containing Openings," derived from their tests at Stanford University as a part of a major study of shear walls sponsored by the Corps of Engineers.

●

Data on both insulating and structural concretes are presented in "Tests of Lightweight Concrete Made with Expanded Blast Furnace Slag Aggregate" submitted by D. W. LEWIS.

"Effect of Axial Compression on Shear Strength of Reinforced Concrete Frame Members," by J. W. BALDWIN, JR. and IVAN M. VIEST, presents results of a recent investigation at the University of Illinois.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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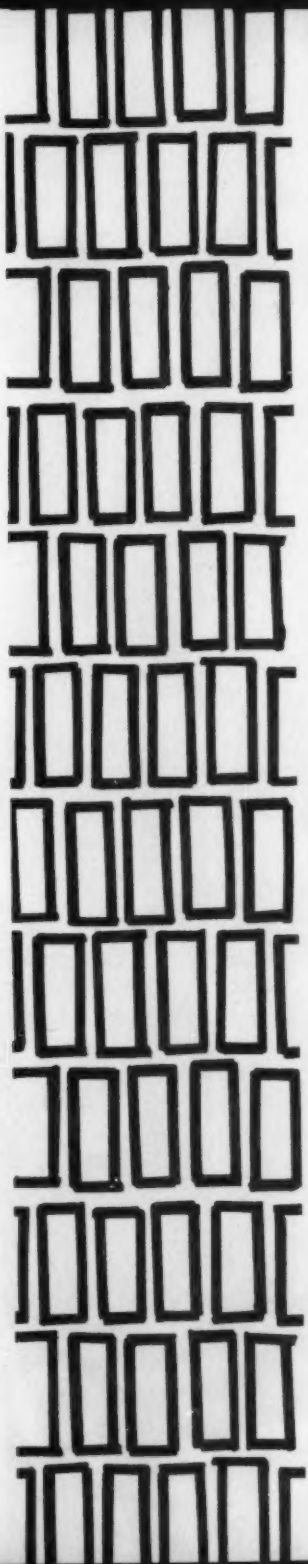

. . . it is impossible to create a beautiful structure in concrete unless the architect be willing to collaborate with the engineer and the contractor from the very beginning. Some of the most astounding results in modern architecture have been obtained by simply having the creativity of the engineer and the contractor, which is of a technological character, help the creativity of the architect, which is of an artistic character.

—*Mario Salvadori*

architectural implications

Title No. 55-25*

ACI headquarters presented a

Challenge in Concrete

By MINORU YAMASAKI†

as told to Mary K. Hurd. The architect for ACI's new headquarters building recounts some of the considerations which influenced his selection of folded plate roof and other design elements. The potential of precasting for economically bringing new form and fine texture to concrete buildings is cited.

WHEN WE WERE FIRST CALLED in to talk to the ACI Building Committee, the problem as presented by the Institute simply was this: A certain amount of new office space was needed by the Institute; the only charge to the architect was to dramatize concrete in an exciting new design. Hitherto we had normally thought of concrete primarily in large structures, so this small building was an interesting challenge.

The ACI building was a sort of turning point; for us it represented not only challenge but education. We wanted to use concrete in an arresting way

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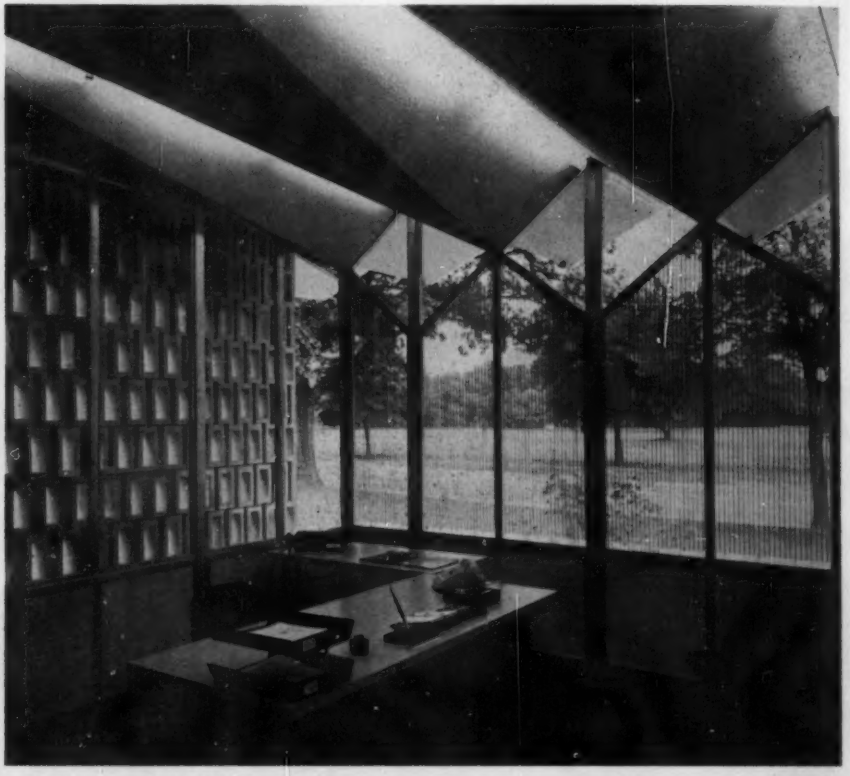
†Member American Concrete Institute, Architect, Yamasaki, Leinweber and Associates, Birmingham, Mich.

CORRIDOR CONTRAST is afforded by X-beams formed where the two folded plate segments of the cantilevered roof meet along the ACI headquarters building's central axis. Skylights are atop the diamond-shaped recesses, triangles of alternating blue and green fill in the other ceiling spaces. Fluting of the 8 in. thick walls was cast in place by fastening wooden inserts to wall forms. Final wall treatment—white paint



STAFF OFFICES, flooded with natural light through exterior glass walls, enjoy the heightened brilliance of white walls and white working surfaces. Fluorescent lighting fixtures are recessed in folds of the concrete roof whose plastered interior surface forms the ceiling for main floor offices. End screen of hollow precast units is decorative here too, and simple lines of well chosen furnishings maintain the free, uncluttered look of interior spaciousness. Year-round air-conditioning assures optimum ventilation

. . . to produce a form of building that was logically impossible in another material. Yet we wanted to keep it looking very light. Prior to the ACI design, the majority of our work had been in steel, but most of our recent work is in concrete.

Cantilevered roof dominates scheme

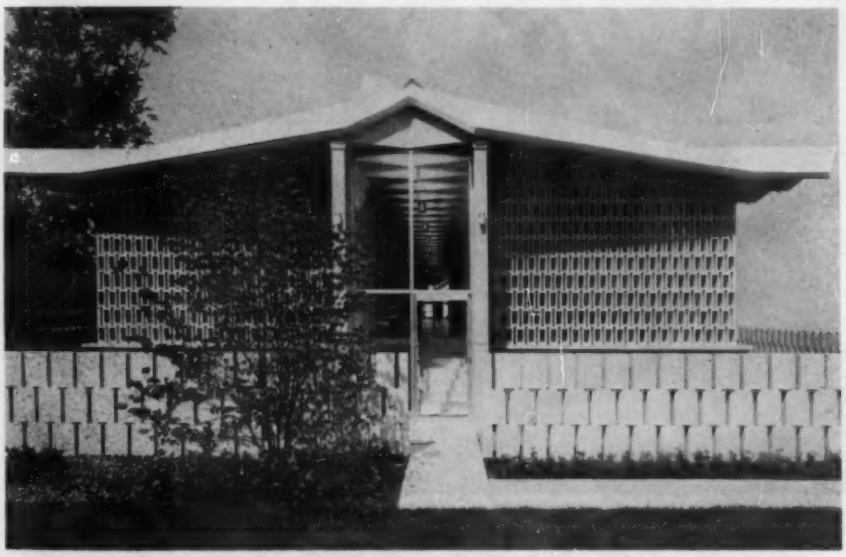
At that time I thought almost entirely of using concrete in planes, rather than in sticks. This was the reasoning behind the St. Louis Airport design; this seemed to be the way of using concrete naturally. Once we had decided on folding the slab, it seemed most appropriate to dramatize it by using the

cantilever in this small structure. Some sort of corridor was required for circulation in the building, and the short span of the roof could easily be cantilevered from the corridor walls.

The problem then was to get the folded plate delicate enough to suit the scale of the structure—the completed roof is essentially 3½ in. thick, except at points where greater cover of reinforcement was needed. Someone suggested using a concrete entrance, but that would have destroyed the cantilevered effect. We wanted to express the cantilever as strongly as possible.

The skylight along the central corridor, which allows a pleasant natural light, is part of the fascinating flexibility of form possible in concrete . . . the roof plane is turned into a beam, which casts its X-shape across the corridor. Triangular skylights nestle at the intersection of the X.

Use of the vinyl coating was a challenge for the concrete industry. From my point of view, it expresses concrete as a material much more than built-up roofing, but this forced us to put insulation on the inside beneath the furring which was surfaced with acoustical plaster. Long lighting troughs placed deep in the valleys of the roof fold complement the abundant natural illumination.



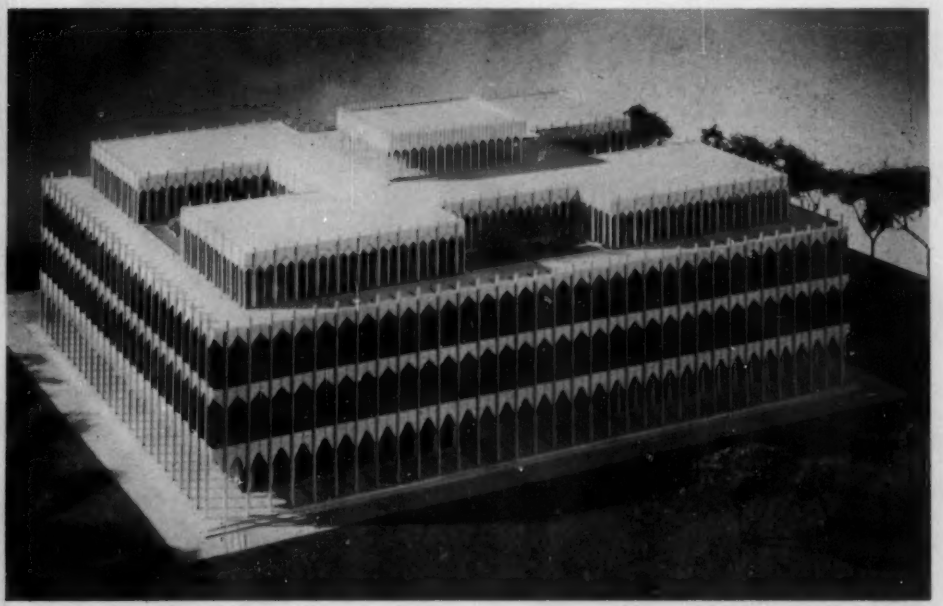
EASTERN ENTRANCE DETAIL is a patterned contrast between surrounding "garden" wall and the non-structural sunshade of hollow precast block. End view gives a better picture of how central corridor walls carry entire load of the roof structure. Block for end screen and garden wall were precast to the architect's specially designed shapes

Other building elements

To maintain accent on the roof cantilever, the exterior walls were necessarily light in treatment. The corridor walls carry the roof load to the basement box which is a stabilizer for the entire structure. Corrugation of concrete in the corridor walls was an ACI suggestion, a good idea though perhaps it should not have been so fine in scale. Here is an intimation of the wonderful things that concrete can do through controlled precasting.

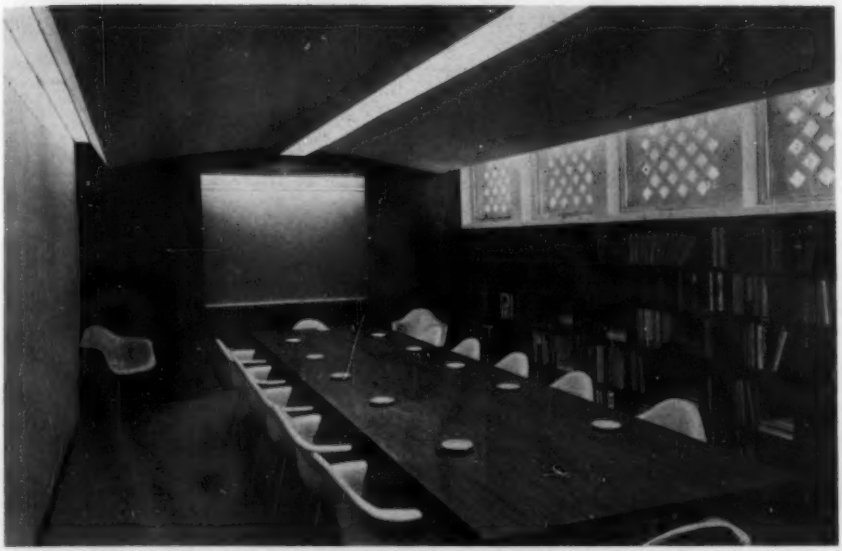
The heating plant stack was kept outside the building, likewise to preserve the freedom of the roof line. Sun shade end walls and precast exposed aggregate panels offered a chance to vary the applications of concrete. The wall around the site is not an enclosing one to keep us away from the neighbors, but one which we thought would become part of the building. Shielding the basement windows with pierced concrete grills expresses solidity at the supporting base of the building where it is most logical.

I was interested in using the hollow core precast units for the first floor since they serve a dual purpose, housing ventilation ducts and utilities con-



Photograph by Baltazar Korab

DAZZLING WHITE of exposed quartz aggregate and white portland cement will dramatize the 40-ft high precast "trees" which comprise the facade of the new home for Wayne State University's College of Education in Detroit. This design by Yamasaki, Leinweber and Associates typifies the adaptability of form available in precast concrete, repeats the architect's favored color in a pattern for continuing campus growth



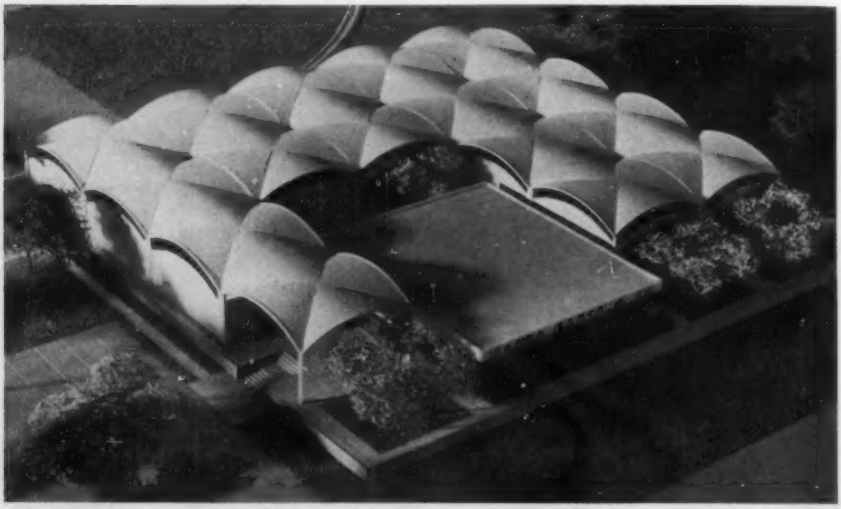
LIBRARY-CONFERENCE ROOM also makes use of corrugated concrete wall surface produced by attaching half-rounds of wood to the face of forms. Diamond pattern pierced in concrete of the precast window grills is visible from building's interior. Dropped ceiling is acoustical plaster. Chalk board can be concealed by sliding panels

duits. Incidentally, in the cast-in-place folded slab we used for McGregor Memorial Conference Center at Wayne State University, circular air ducts are inside the fold.

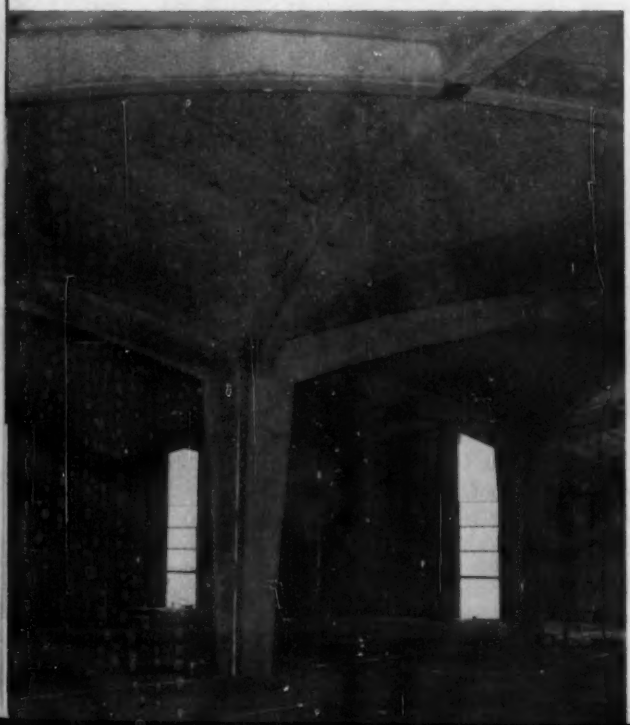
Color at ACI

Color for the Institute headquarters was used sparingly with the total architectural effect in mind. So far I am not very keen about color; color will never mean much to me unless it can make concrete more attractive in its natural state. Colors become muddy when mixed with gray cement. Perhaps color should be mixed only with white cement concrete to attain the clarity so essential to colored materials.

Most of the exterior concrete has been painted gray, lighter and warmer than the natural color of concrete. Exposed interior concrete is painted white. The roof, of course, is coated with white plastic, and the precast panels at the base of the window mullions have rose and gray quartz set in a base of white mortar. A polished black terrazzo stair railing accents the front entrance and touches of black recur throughout the interior. Pale aggregate in a white mortar base comprises the terrazzo floor of the lobby, is repeated in stairways and landings. A single surprise of bright color is in the corridor ceiling where recesses in the X-beams have been painted with alternating triangles of blue and green, joining in a pattern with the varying hue of the skylights.



SIXTEEN BAYS, 40 x 40 ft, roofed with intersecting precast shells comprise the warehouse section of a structure designed by Yamasaki, Leinweber and Associates for Parke-Davis. The roof requires only six interior supporting columns made up as shown in the view below by joining together the vertical legs of L-shaped precast rigid frames



PRECISION PRECASTING at Parke-Davis warehouse and office building, Menlo Park, Calif., simplified construction, left clean, exposed concrete interior. The 3½ in. thick triangular roof shell components were cut in half for easy erection. Rigid bent members, of modified L-shape, locked together diagonally acorss 40-ft bays. Side walls are also precast. No centering needed for field erection

Precasting implements new form in architecture

The notion of precasting came after the design at ACI, and was not an inherent part of the idea. Mr. Corbetta, I believe, suggested precasting, and the scheme was most intriguing. An alternate for precasting the roof was included in the bid proposal. The contractor chose to field cast the roof elements, and although there were many problems with this method, it was better, I believe, than cast-in-place work would have been.

I have become intensely interested in precast concrete. This pioneering work for the Institute has opened the way for major developments in the industry. Through precasting concrete gains mechanization in its production, and through the processes of precasting and prestressing we can bring new form to concrete for architecture. It offers a promise of escape from the flat, boxy architecture that has been running rampant over the country. The concrete industry lags behind in its awareness of this potential; similarly it has scarcely begun to imagine what it can offer architecturally through the advantages of shop control in producing fine texture.

Photograph by Baltasar Korab

SHOTCRETE SURFACE treatment on all structural concrete at McGregor Memorial Conference Center, Wayne State University, Detroit, developed precision of finish to meet exacting architectural requirements. Cast-in-place folded plates span about 40 ft, and ventilating ducts are cast within the folds. The columns are faced with white marble

Realizing some of this has caused me to shift my earlier view that concrete was inherently more natural in slabs and planes. Our Education Building for Wayne State University will use 40-ft high tree shapes precast in white surfaced concrete, as the accompanying illustration shows. The work we have done for the Parke-Davis office and warehouse at Menlo Park, Calif., has been a most satisfying experience with precast concrete. L-shaped column and roof support bents are used there with spherical-triangular roof shells and flat wall panels.

Elegance in concrete

Precast concrete is the most exciting of structural materials—it is the future of structure. This is true not only in dramatic spans, but in ordinary, everyday buildings.

My major indictment of the concrete industry is that it persists in thinking of its product as a *crude* material. No building should be built crudely; all good architecture throughout history has been fine and elegant. This can and should be attained with concrete, and with precast concrete both elegance and economy are possible. We as a society should be mature enough to provide ourselves with beautiful surroundings. Beauty is not a matter of cost; its attainment rests on discrimination in selection.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.

Title No. 55-26*

CANTILEVERED FOLDED PLATE

roofs ACI headquarters

By CHARLES S. WHITNEY†

THE STRUCTURAL ENGINEER REPORTS on some of the problems he considered in designing the folded plate roof which cantilevers 20 ft from central corridor bearing walls at the new ACI headquarters building. Illustration and brief comment show roof in relation to the total structural scheme.

In spite of the fact that the headquarters building of the American Concrete Institute is one of the most unusual and exciting structures in the country, the basic concept is so simple and direct that very little need be said about the structural design. The original concept was presented by Mr. Yamasaki, the architect, in the form of a model to the ACI Building Committee about 2 years ago. It was approved immediately and has been executed with no substantial change.

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†Member American Concrete Institute, Consulting Engineer, Partner, Ammann and Whitney, New York, N. Y., and Milwaukee, Wis.

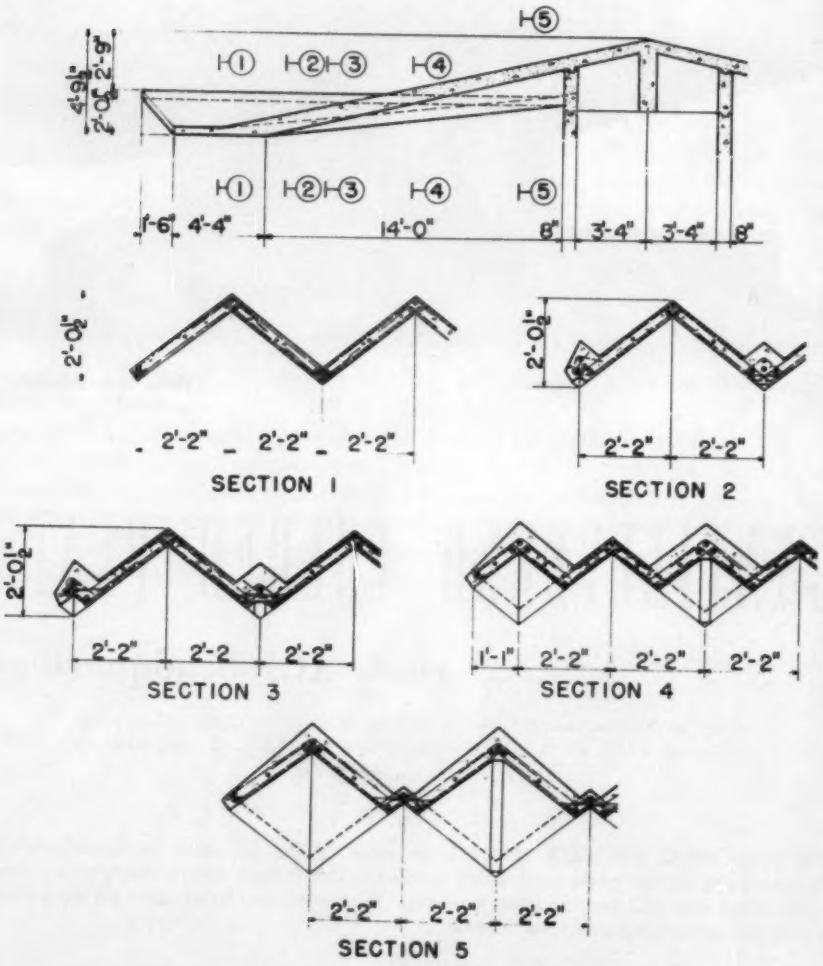


Fig. 1—Cross sections of roof based on original cast-in-place proposal with one detail (bottom) of precast alternate which was finally adopted by contractor

The writer has had the privilege of collaborating as structural engineer with Mr. Yamasaki for a number of his buildings. Each one shows originality and a keen understanding and appreciation of the characteristics and potentiality of reinforced concrete. In each case, the structural forms have been suggested by the architect and involve the most modern techniques.

Structurally there is nothing unusual about the ACI building except that the roof is cantilevered out some 20 ft from the corridor wall and receives no assistance from the window mullions. This was at the insistence of the architect who would tolerate no compromise. Too much familiarity with the effects of plastic flow tend to make one conservative, and it was suggested that the mullions might be snuggled up under the cantilevers after the dead load and early plastic deflections had occurred. This idea was not accepted and presently there is a space at the top of the exterior walls and interior partitions intended to accommodate all future deflections. The cantilever is therefore a true, simple cantilever and not a fixed beam with an indeterminate reaction at the outer end.

The deflection at the cantilever ends was about $\frac{3}{8}$ in. due to dead load when the falsework was removed. Plastic flow and additional dead load have increased it to about $\frac{3}{4}$ in. at this time.

The cantilever shell was designed by the ordinary beam theory (straight line variation of strains) because the proportions of the plates make the more accurate folded plate theory unnecessary. The repeated unit (Fig. 1) is 4 ft 4 in. wide, about one-fifth of the cantilever span. At some sections the plates are thickened beyond the minimum of $3\frac{1}{2}$ in. to make room for the required reinforcing steel and for architectural reasons to improve the external appearance.

In the direction normal to the cantilever span, the plates are reinforced as continuous slabs supported at the folds, and reinforced for both positive and negative moments to provide for nonuniform or concentrated loads. They are also reinforced to provide for 100 percent of the beam shear. The plates are relatively thick and the stresses near the free edges are not severe.

Because of the small size of the building, the roof was detailed to be cast in place, but the specification permitted the contractor to use precast units subject to the architect's approval of his suggested details. The low bidder did elect to precast the roof in units 4 ft 4 in. wide which were welded together after erection to provide continuity of the plates.

Lightweight concrete would have been desirable because of its greater insulating value, but was not used because of its higher cost. Because of the complicated surface of the roof, insulation and roofing were also too expensive, and the outer surface was given a coat of plastic, with the under surface furred and plastered. This will force the concrete shell to follow the range of external temperatures with resultant flexing of the slab which may or may not cause some trouble in the future. In any case, it increases the importance of continuous top and bottom steel in the slab to toughen it and to resist flexure.

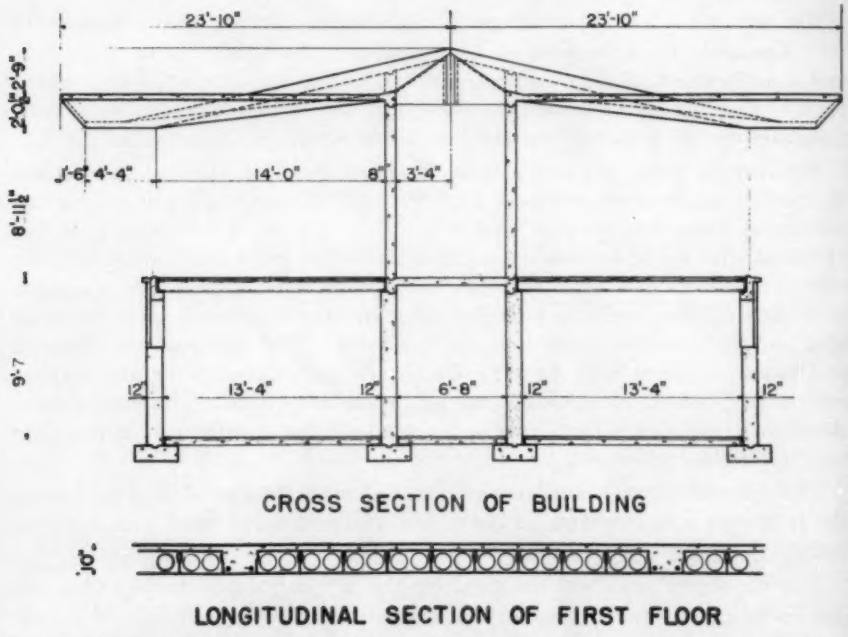


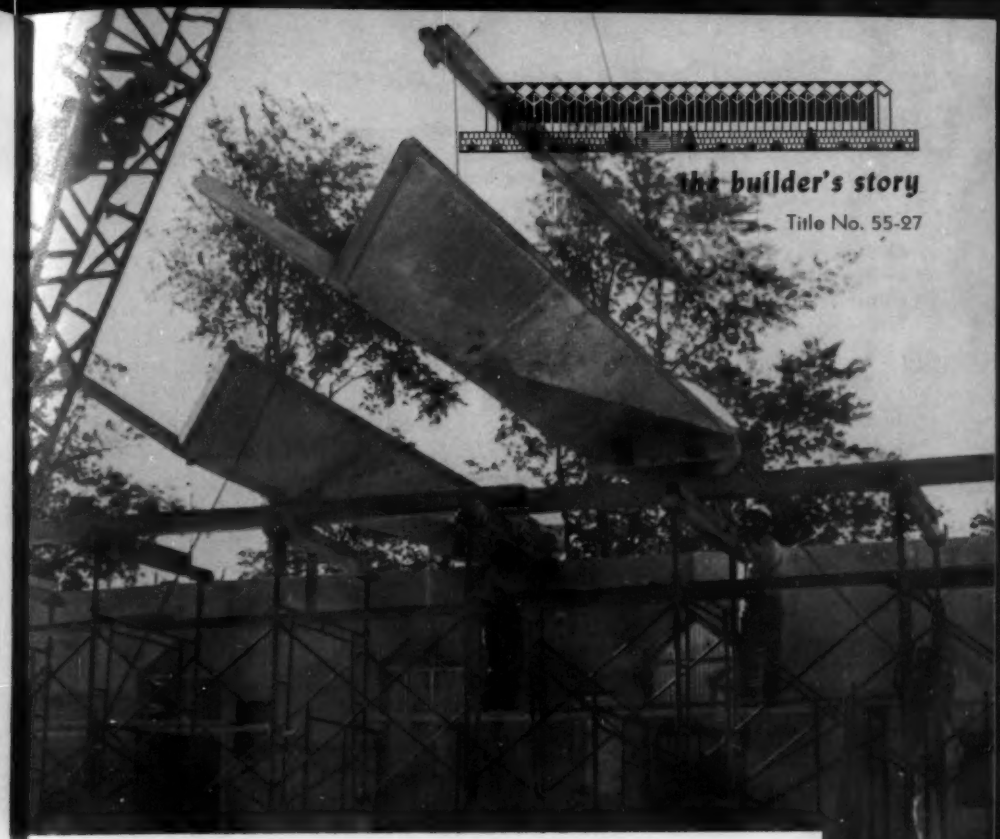
Fig. 2—Longitudinal section of first floor, and cross section of the entire building showing rigid box which comprises the basement structure

The roof cantilevers are supported by the two monolithic reinforced concrete corridor walls which are themselves reinforced as cantilevers extending up from the basement corridor walls. They carry lateral forces to the first floor, below which the entire building is a rigid box (Fig. 2).

Basement exterior and corridor walls are reinforced concrete. The first story corridor floor is a solid concrete slab. The balance of the first floor is made of 8-in. hollow core units on a 13 ft 4 in. clear span with cast-in-place beams of the same depth, spaced about 13 ft. The hollow core units were used to provide ventilating ducts leading to the exterior wall in the first story. The concrete floor finish over the precast units would have covered the floor outlets more thoroughly if it had been at least 2½ in. thick instead of 2 in., but it has been reasonably satisfactory to date.

Although this building is of considerable interest because of its architecture and construction methods, there appears to be little to be learned from it structurally except how satisfactorily a folded plate without external insulation may serve as a roof of an office building in a northern location. Plastic coatings have been used on a number of concrete folded plate roofs and so far the experience is not discouraging.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.

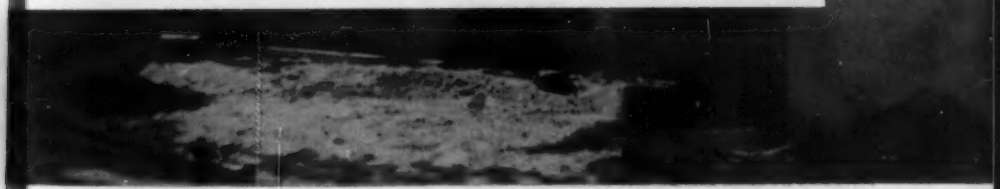


the builder's story

Title No. 55-27

Construction for ACI*

By JOHN STRANG†



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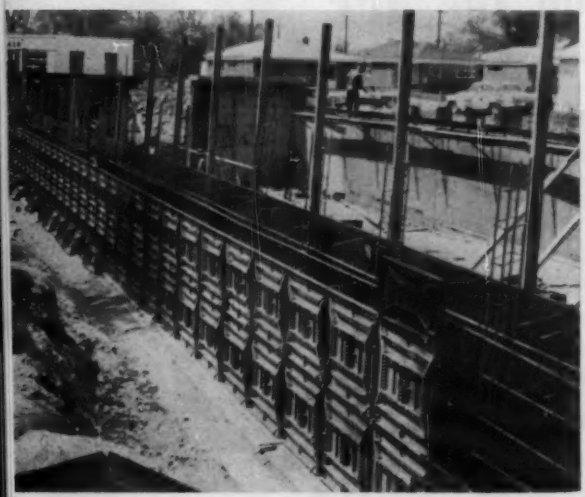
†Member American Concrete Institute, Vice President, Pulte-Strang, Inc., Ferndale, Mich.

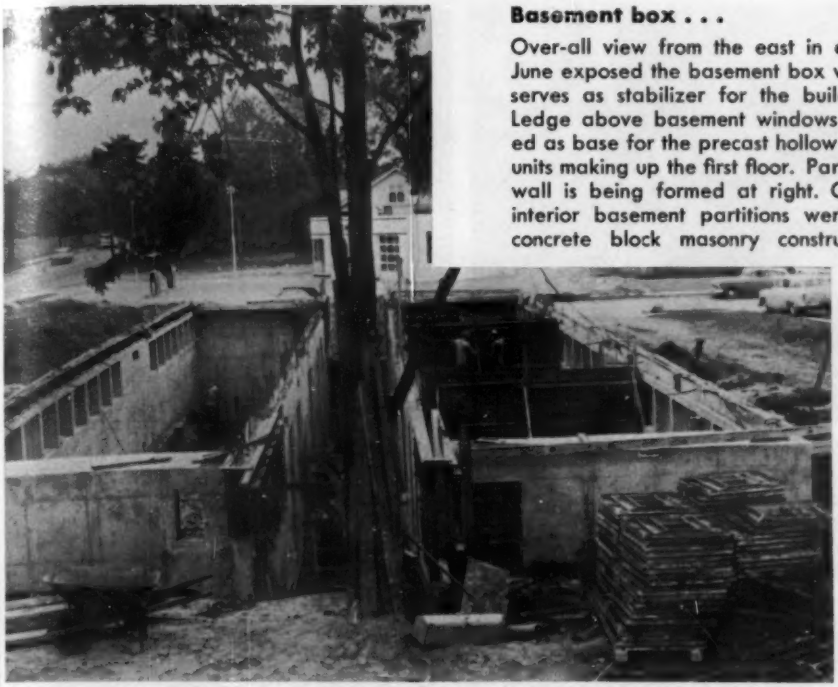
The general contractor tells of precasting and erection of 46 folded plate sections which comprise the roof of the ACI headquarters building. Concrete mix data are given, and problems of establishing an architecturally satisfactory finish are cited. Cast-in-place fluting of the corridor walls is also discussed.

■ EXECUTION OF THE UNIQUELY designed ACI headquarters leaned heavily on good, conventional construction practices, but there were some unusual and difficult problems for the contractor. Aside from providing good housing for the Institute staff, the building serves the dual purpose of displaying concrete advantageously in its many varied forms. The cantilevered folded plate roof, precast on the job, the thin exposed aggregate panels, the decorative grills precast to shield the basement windows, fluting of the cast-in-plate corridor walls—these and other applications of concrete all had to be of top quality, and the contractor was responsible for their strength, durability, and perfection of surface finish.

Concreting was in full swing as the 1957 construction season opened. Late in March, general contractor, Pulte-Strang, Inc., erected a field office, cleared the construction site, and began foundation excavation. Unfavorable soil conditions appeared during excavation, and it was decided to lay the north wall footings and corridor footings on blue clay soil to avoid the unpredictable wet fine sand. Footings are of varying heights, and continuous footing reinforcement consists of four #5 bars. A keyway was formed in the footing to tie into the walls.



Steel forms for the south wall being put in place following stripping of the north wall (being wet down in background). Steel is in place for columns between basement windows



Basement box . . .

Over-all view from the east in early June exposed the basement box which serves as stabilizer for the building. Ledge above basement windows acted as base for the precast hollow core units making up the first floor. Partition wall is being formed at right. Other interior basement partitions were of concrete block masonry construction

WALL CONSTRUCTION

In May forms were erected and placement of concrete began for the basement walls. Standard steel panels 30 x 30 in. with numerous special sizes as required made up most of the forms with special attention going to the columns and beams between and above the basement windows on north and south walls. Vertical and horizontal reinforcement, #5 bars, was placed at 10 in. on centers in all of the basement walls. Form layout on all exposed walls was planned for symmetrical appearance. The resulting smooth surface required patching only at the tie holes and finishing to fill surface air bubbles. Exceptionally neat sharp corners and alignment were achieved as a result of the tight forms.

Fluted walls

In the library-conference room the architect specified random sized half-round recessed fluting. Milled lumber half-rounds were attached to the metal forms by sheet metal screws. The results, although acceptable, required extensive finishing work by the contractor and painter.

Fluting of the first floor corridor walls projects ½ in. from the wall surface and is trapezoidal in cross section. Plastic faced plywood forms with milled wood fluting pieces attached were used. Since air temperatures ranged up to

Reinforcing for roof segments was assembled on a jig of exact size and shape of the folded slab form; three #9 bars are main reinforcement for each roof segment

Detail of reinforcing and form for beam box which was cast at the interior end of each slab section. Two boxes form X-beam that spans the central corridor

100 F, a retarding admixture was used to avoid "cold joints." In spite of the high quality form material and careful form workmanship, careful casting, and vibrating, it was impossible to meet the architect's finish requirements without much hand labor in patching corners, grinding, and finishing surface air bubble marks.

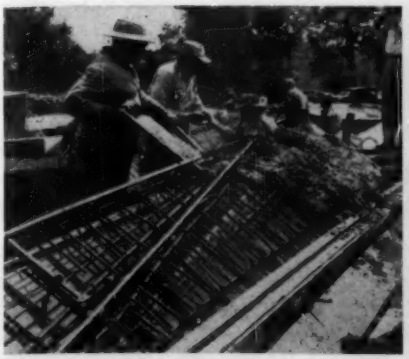
Architects who desire extremely fine surface and sharp corners in decorative concrete fluting might cut finishing costs and avoid difficulty by planning their structures so that decorative wall areas can be precast horizontally. Precasting the corridor walls for the ACI office was not considered feasible with a precast roof due to difficulty in making structurally continuous joints at floor and roof.

FLOORS

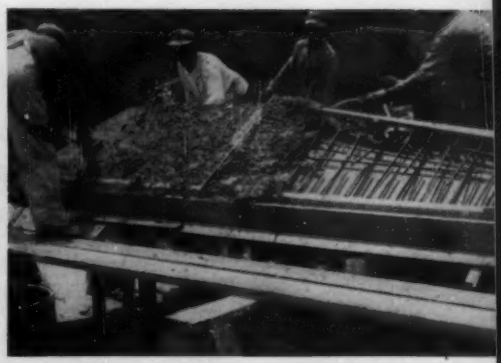
Basement floors were cast over a 4-in. base course of compacted bank run gravel and a 4-mil plastic vapor barrier. The 4-in. slab was reinforced with wire mesh. Hollow core precast units were used for all of the main floor except the cast-in-place corridor floors. Heating, ventilating, and wiring installations were made in the hollow core of floor members. A 2-in. topping—made with 1 part cement, 2.5 parts sand, and 2.5 parts pea gravel—was used above the precast floor units and on the basement floor slab.

ROOF CONSTRUCTION

The plan showed a reversed fold, folded plate roof cantilevered from each corridor wall. Deep X-shaped beams with diamond-shaped openings for skylights formed the tie across the corridor. The design was for cast-in-place construction, but precasting was permitted at the contractor's option.



Placing concrete in form was by shovel from elevated platform where it had been unloaded by truck mixer. Vibration followed, ahead of finishing shown at right

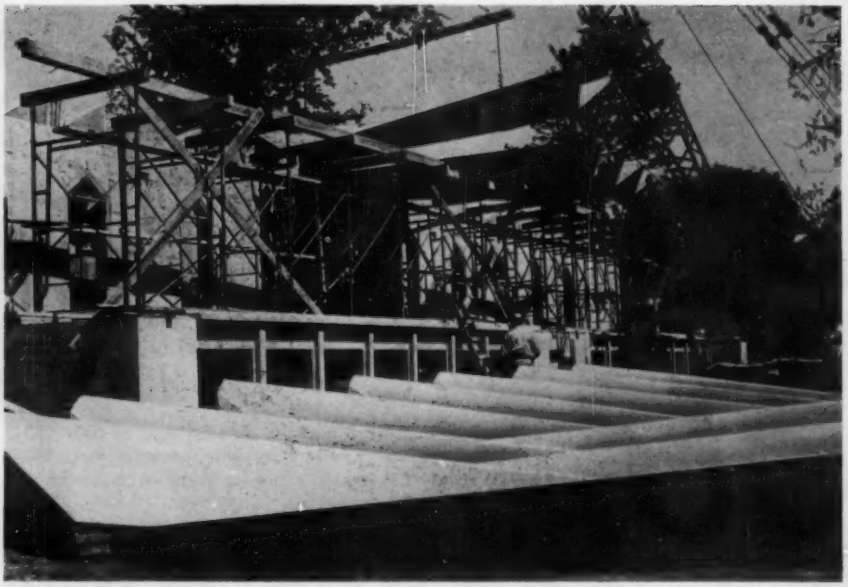


Screeding and troweling provided surface finish. Portion to right was troweled since it was to be exposed in roof overhang; remainder was screeded only, later covered by interior plaster on the ceiling

Knowing that the architect set high standards for finish appearance, the contractor decided to precast the roof principally because precasting the sections upside down permitted achieving sharply formed lines for the exposed upper roof surface. Plastic faced plywood was used for three forms built on and anchored to substantial footings for on-the-job precasting. A dummy mock-up form was used as a pattern for cutting the plywood and later served as a jig for assembling all reinforcement in units. An elevated platform was built beside each form so that truck mixers could pull up and discharge concrete on it.

More than 70 detail shapes of reinforcement were used, with three #9 bars forming the main longitudinal reinforcement of the 23½-ft sections. Steel plates ⅜ in. thick along each edge of the section formed the transverse joint detail developed by the contractor. Two rows of dowels spaced 4 in. on centers were welded to these metal plates and embedded in the slab. After

Finishing touches . . .
Final finishing of roof slab segment in the foreground, with reinforcement in place for casting the next slab on the adjacent platform. At the right, rear, is a third section awaiting lifting from the form. Finished sections of the roof are seen in the background



Erection sequence . . .
Slabs were first placed on the ground near their approximate position in the building, then hoisted individually to rest on temporary scaffolding, later welded for continuity

erection of the sections these plates were welded top and bottom for moment resistance, and longitudinal reinforcement was welded along the corridor center line where front and back halves of the roof joined.

Concrete mix for roof slab

Concrete for the folded plate members had to meet severe requirements. First, the varied shapes of reinforcement and inserts in the $3\frac{1}{2}$ in. thick slabs made placing difficult and slow and required a $\frac{3}{8}$-in. maximum size aggregate. Due to the July heat at time of casting, a retarded set was required. A casting schedule calling for two units per day required 1500-psi concrete at 24 hr, and 4000 psi was the average strength specified. A mix containing $6\frac{1}{2}$ sacks of Type I cement plus a cement dispersing, water reducing admixture and pea gravel aggregate was employed. With an average initial slump of 3 in. the set was retarded for approximately 1 hr at concrete temperatures up to 95 F. One-day strengths averaged 2100 psi. Initial curing under wet burlap for 24 hours was followed by a sprayed on liquid membrane-forming white pigmented curing compound. The 28-day strength averaged 5100 psi.

Erection of the roof slab units

After 28 days curing, all units were turned over for clean down on the top side. Threaded eye bolts for lifting and threaded socket inserts set flush minimized patching of the top. All units were numbered and measured to

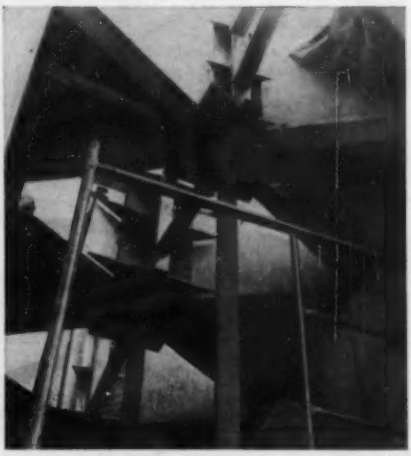
determine over and under width tolerances. An erection sequence was prepared to avoid cumulative overrun. In spite of all precautions a few transverse plates had to have opposing bulges trimmed by a cutting torch to secure the close fit required.

Roof units were erected as simple beams on falsework at the window line and the corridor bearing wall. Two steel shims 4 in. high on the bearing walls plus the point support on falsework gave a stable three-point bearing that permitted adjustment to match adjoining units. After completing main welds above the corridor and tie down welds to dowels in the bearing wall, nonshrink grout was used to complete the center joint and bearing on the wall.

When structural continuity of the roof had been established, shoring was removed. A camber of 1¼ in. was provided at the window line, but immediate deflections averaged 5/16 in. The windows are fixed by a sliding joint to allow for future live load and creep deflections. Transverse roof joints above the welds were completed to sharp edges with a patching grout containing a polyvinyl acetate emulsion as an admixture and as bonding coat. The exposed roof slab is covered only by a 40-mil thickness of vinyl membrane coating sprayed on in five layers, a treatment that retains the architectural integrity of the concrete surface.

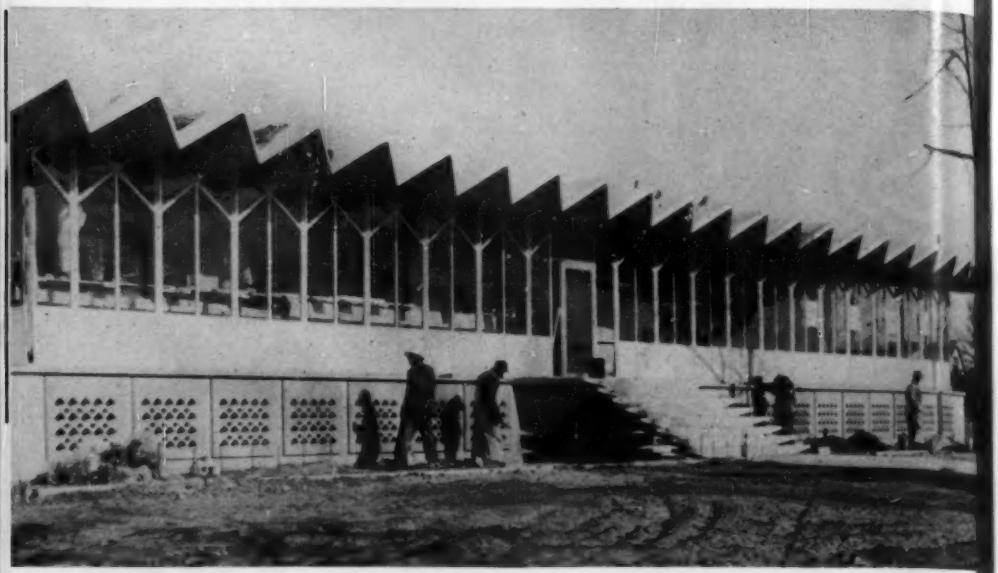
Central corridor with all roof segments in place ready for welding of the longitudinal reinforcement at middle of the hall which joins the triangular beam boxes

After welding, forms were set in place (below) for casting the short connecting member in nonshrink mortar



MATERIALS

The specifications required all aggregate for exposed concrete to conform to ASTM C 33 limits on deleterious material. In the Detroit area most natural gravels contain deleterious



Nearly finished . . .
Front facade assumed a finished look when glazing was completed. Here workmen made a check of final clean-up on precast grill slabs which shield basement windows

material which causes "popouts" after freezing and thawing. Therefore, premium aggregate produced by heavy-media separation was used in the concrete for roof, building walls, and paving.

All exterior concrete was painted with a water-thinned polyvinyl acetate emulsion paint. This type of paint was selected to provide a "breathing" coating which would not be too absorbent to contamination by industrial smoke and soot.

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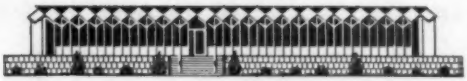
Concrete reinforced with fibers of steel is the body of our modern world in any form. Concrete with steel fibers embedded in it, which is very like your own structure or the structure of a tree or any structure nature indulges in, is going to be the body of our modern world.

—Frank Lloyd Wright



design and construction

Title No. 55-28

Folded Plate Dome Ideal for Auditorium*

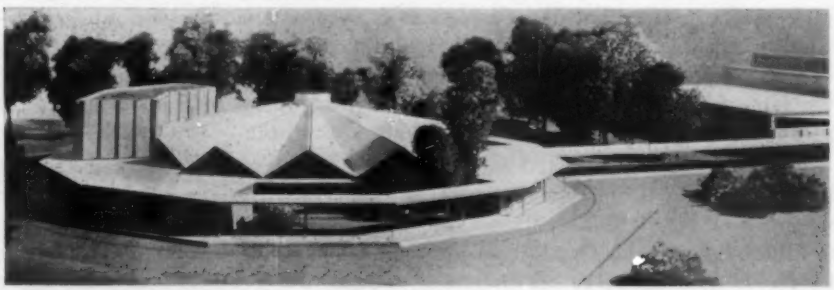
By LYNDON WELCH†

Design and construction of a folded plate roof over an auditorium at Wayne, Mich., are discussed. Dimensions, design loading, decentering procedure, and deflection are described.

DESIGN REQUIREMENTS

The Wayne Auditorium at Wayne, Mich., completed in the spring of 1958, is an example of unusual design and careful construction in reinforced concrete.

The relatively wide pattern of the 900 seats falls naturally within the outlines of a circular enclosure approximately 100 ft in diameter. The stage



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shop, dressing rooms, band rooms, and public spaces, which make up the balance of the project, fit into a peripheral structure of moderate scale (see photo above and Fig. 1). The stage house is the only structure outside of the central space to rise higher than the proscenium.

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†Member American Concrete Institute, Chief Structural Engineer, Eberle M. Smith Associates, Inc., Detroit, Mich.

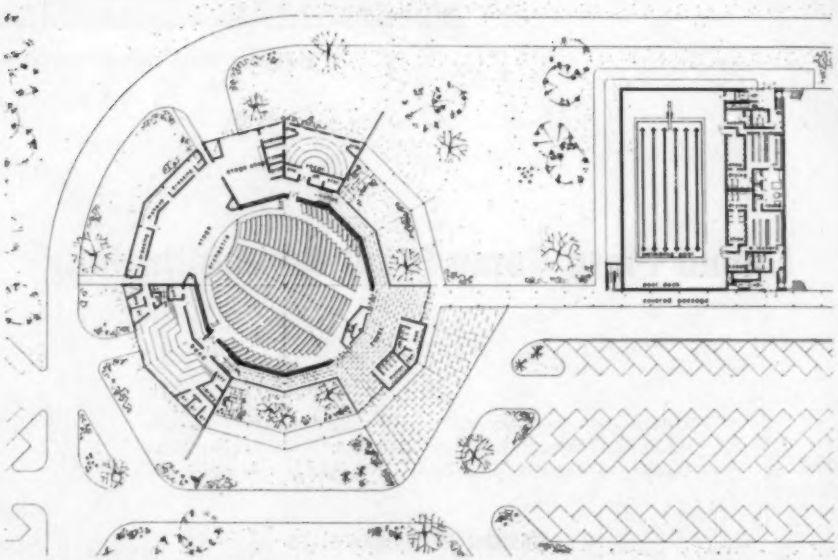

Fig. 1—Pattern of the 900 seats falls naturally within the outlines of a circular enclosure. The stage shop, dressing rooms, band rooms, public spaces, and stage house are grouped around the central auditorium space

It was necessary to devise a roof over the auditorium consistent with and expressive of its character and function. The most direct solution to roofing a circular area of such a size is some sort of dome. A surface of double curvature, such as a dome, ordinarily creates difficult problems in the fabrication of structural steel or in the formwork for reinforced concrete. What is worse, the curvature tends to focus sound so as to produce loud spots and dead areas which would be objectionable in an auditorium.

These considerations led logically and naturally to the use of a "folded dome" made up of intersecting planes. The basic structural design was for a reinforced concrete folded plate, for which the formwork would be a series of tilted flat panels, faced with plywood. The folds would tend to break up and scatter sound rather than concentrate it.

During negotiation of the contract, an alternate design for the same shape was prepared in structural steel. When estimated, however, the difference in total construction cost was slight, and favored the reinforced concrete design.

FOLDED PLATE ROOF

As actually built, the roof is a 12-sided polygon composed of 12 V-shaped plates spanning 44 ft and rising 15 ft from a perimeter tension ring to a central compression ring. The tension ring is made up of four structural steel angles

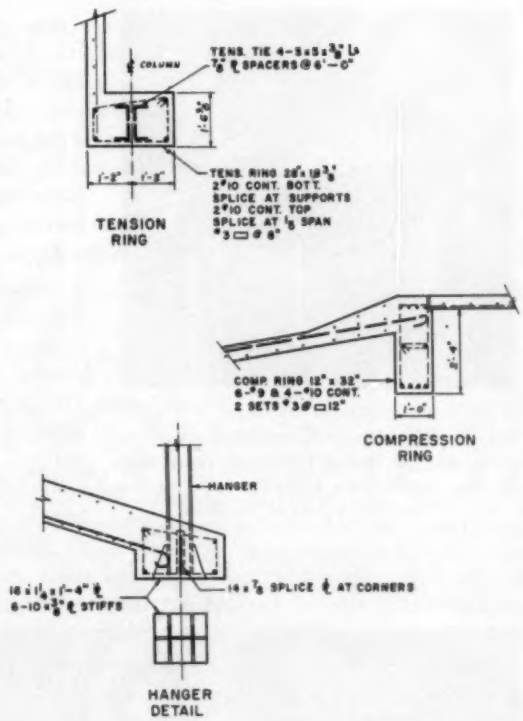

Fig. 2—Construction details. Twelve V-shaped plates span between the tension ring (structural steel) and the reinforced concrete compression ring. The 4-in. cap slab is isolated by expansion joint material from the compression ring and does not participate in its structural action

bolted to bent plates at the vertices of the polygon. The tension steel is encased in concrete, and concrete diaphragms fill the triangular spaces between the tension ring and the folded plates. The roof is supported on 11 round concrete columns and one structural steel hanger from a heavy steel truss above the proscenium. Details are illustrated in Fig. 2.

The V-shaped plates are not of a uniform thickness. At each ridge the thickness is about 4 in. At each valley the dimension varies from about 5½ in. near the center to nearly 12 in. near the supports where the additional cross-sectional area is needed to transmit thrust to the tension ring. The variation in thickness and the fact that the plates are cantilevered several feet beyond the diaphragms, tend to shift the center of gravity of the plates outward, reducing the thrust to some extent. The plates are also thickened locally near the compression ring to reduce shear stresses and to provide more room for reinforcing steel.

In analyzing the plates for combined axial load and bending moment it was necessary to set up expressions for the characteristics of a nonrectangular beam of varying cross section. Since the cross sections varied linearly with distance from the center, it was possible to express these characteristics as a group of definite integrals; this procedure made the calculations much simpler.

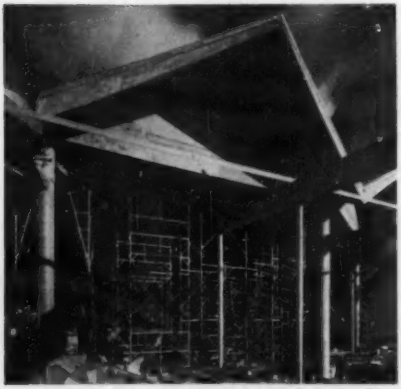

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Fig. 3—The formwork consisted of ¾-in. plywood over timber falsework supported on steel scaffolding from the level of the tension ring down to the floor

Two basic loadings were assumed; 30 psf live load on the entire roof, and the same live load on one-half of the roof. It did not appear that lateral wind load would produce any significant stresses within the structure; however, the stage tower was heavily braced against lateral load.

In general, the analysis followed the standard procedure for folded plates. Slab elements were assumed to span from valley to ridge; they were considered continuous at the valley, but simply supported at the ridge, where construction joints were to be permitted. The V-shaped elements were analyzed as homogeneous, isotropic beams of varying cross section, and all stresses were referred to the centroidal axis. This approach appeared quite reasonable in view of the fact that principal stresses throughout the most highly stressed region were compressive.

The beams were initially assumed simply supported at the compression ring and at the tension ring, and were then checked for fixity at the compression ring by a method for which the author is indebted to Alfred A. Parme of the Structural Bureau of the Portland Cement Association, and which takes into account the angle change of the ring under applied external moment. Assuming linear distribution of strain from top to bottom of the ring, Parme obtains the following expression for the angle change:

$$\Delta\phi = \frac{12\,Mr^2}{Eth^3}$$

In this expression:

M = moment across diametral section of ring
r = radius of ring
t = thickness of ring section
h = height of ring section

With this expression, the stresses in the compression ring and in the beam elements as modified by continuity at their juncture may be computed by standard methods.

Since each ridge and each valley lies on a line of symmetry, there is no significant longitudinal shear transfer between plates, even under unbalanced live load.

Calculations were made by both conventional and by ultimate strength methods. On the basis of a load factor of 2 for the latter method, there was fairly close agreement in the results.

The major reinforcement was concentrated at ridges and valleys and near the tension and compression rings, where the bars were designed to take all the shear. Transverse slab reinforcement was generally established by the minimum ACI Code requirement.

CONSTRUCTION

The roof was formed on timber falsework faced with ¾-in. plywood and supported on steel scaffolding from the level of the tension ring down to the floor (Fig. 3). Although the contact surface of the formwork for each panel was flat, the pitch in the direction of maximum slope was 31.6 deg, and the contractor faced the problem of establishing a pattern of supports and of laying out a large number of oblique cuts. Consequently, he obtained approval of details of construction and support of a full scale panel of formwork in his yard before pre-cutting materials for shipment to the site.

Although the design of the roof would have permitted the use of movable formwork supported on a central tower, the contractor elected to place all of the concrete in one operation. He used two 100-ft cranes, each working

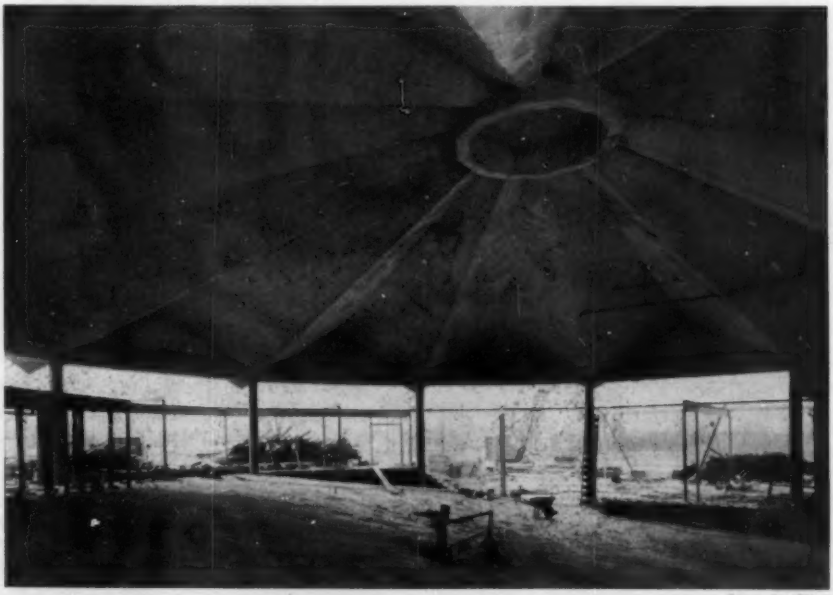

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Fig. 4—Formwork was decentered at 19 days. Deflections were continually measured by a simple gage. Perforations in the lower surfaces of panels serve as a plaster key. The subframes at the upper right are to support spotlights illuminating the stage

with a pair of buckets and succeeded in placing the 300 cu yd of transit-mixed, standard weight concrete in about 9 hr. The cranes worked in opposite panels, lifting one bucket while the other was being filled, and moving counter-clockwise as soon as a panel was complete. Studies had been made with a small scale model to establish crane locations and clearances.

Even under continuous vibration, the 2-in. slump concrete showed little tendency to travel down the slopes, probably because the typical reinforcement pattern consisted of a large number of small bars fairly closely spaced. The two six-man teams of finishers had their problems, however, since they had to screed and trowel while backing "up the hill." The screeds were left in place for later attachment of built-up roofing.

The roof was protected with burlap for 14 days. At the end of this period, tests of field-cured cylinders indicated that the concrete had reached its design strength of 3750 psi.

The formwork was decentered at 19 days. Considerable thought and discussion had been devoted to establishing procedure for this last critical phase of the construction. The actual operation turned out to be almost disappointingly simple. Each of six men was assigned a sector of scaffolding and, on signal, jacked down the legs one turn, (about $\frac{1}{8}$ in.) working uniformly from the center toward the perimeter. The process was halted from time to time, while the deflection was checked with a gage (Fig. 4) anchored to the compression ring. When every support had been loosened, the six-man team went through the pattern again, jacking down two turns this time. After this, all jacks were loose. The formwork was left in position for 24 hr and then dismantled.

The maximum anticipated deflection at the center of the structure had been calculated as 1 in. The calculation had purposely ignored the restraints furnished by the diaphragms and the concrete columns, and was considered to be conservative; it was. In 24 hr, the gage deflection was $\frac{5}{8}$ in. Since the gage has a multiplying factor of five, the true deflection was $\frac{1}{8}$ in. Although the gage was left in place for several weeks, no significant change in deflection was observed.

ACKNOWLEDGMENT

A. Z. Shmina and Sons Co. was general contractor for the Wayne Memorial Auditorium. Eberle M. Smith Associates of Detroit were the architects and engineers.

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Title No. 55-29

Mr. Samuely discusses design principles for folded slab roofs, stair landings, and galleries, with special attention to shear forces and to stresses produced by differential deformation in the slabs. Beam action of the fold or "quoin" is explained. Precast elements, prestressing, and composite construction (precast with cast-in-place concrete) are shown as they apply to folded slab construction. Structures in England and America illustrate use of folded slabs in a wide variety of shapes.

Folded Slab Construction*

By Felix J. Samuely†

When shell construction was first developed about 30 years ago, the intention was to evolve a method by which large areas could conveniently be covered with long span constructions, using a minimum of material. The principles of design were first established for a concrete of almost infinitesimal thickness; later necessary additions were made to allow for the fact that every shell construction has to have a certain thickness which in some cases may be quite substantial, and which can have a great influence on the flow of stresses. Concrete roofs have always been in competition with lightweight materials like cement asbestos, bitumen-covered corrugated steel, and others, and it appeared necessary to develop a concrete roof of as little weight as possible, particularly as this weight constituted the greater part of the total load.

At that time even the cost of manufacture was considered completely subordinate to the weight. Since then, the experience of 30 years has directed shell construction into certain lines. Because of the rather expensive form-

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†Consulting Engineer, London, England.

work, this type of construction can be used economically in two instances: (a) where there is sufficient repetition to insure considerable reuse of formwork (the shape of the shell most suitable for the particular application can usually be found); and (b) wherever standard formwork can be used, the reuse of which is not dependent on the building in question.

The foregoing restrictions allow shell construction much less application than its principles would deserve. They do not permit economical competition in such instances as small factories unsuited to standardized construction, nor for theaters, assembly halls, market buildings, etc. where in fact the principle is valid just as much as in the type of buildings described in (a) and (b). Furthermore, the shell which is known as a series of vaults following each other does not allow a great variety of shapes. (The only practicable variation is a circular or elliptic dome.) When it loses the attraction of novelty it can become a source of architectural boredom, and this may already be noticed in some of the new towns in the south of England, where acres and acres of industrial buildings are being erected with no variety in outline or in plan.

SUBSTITUTING THE FOLDED SLAB FOR SHELLS

Where shell construction is not economical, or where its application is undesirable for other reasons, the principles of its construction can be retained with folded slabs. If a cross section of a shell is drawn and the continuous curve replaced by a series of straight lines, the structure obtained is now commonly called a folded slab (or folded plate). In a larger sense a folded slab is any series of slabs, rectangular or otherwise, which are supported and composed in such a way that they form a stable structure.

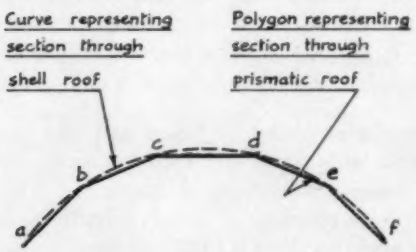


Fig. 1—Replacing curve by a series of straight lines

Shells require diaphragms for stability, generally in line with the columns, and the same type of diaphragm is required for a folded slab, with the one variation that if a folded slab consists simply of a series of zig-zag frames the diaphragm can be replaced by a simple tie member from eave to eave.

In replacing a curve (forming a cross section through a shell roof) by a series of straight lines, nothing is altered in the principle of the structure, but local bending moments are set up; for instance, in Fig. 1, the slab must be able to span from *a* to *b*, from *b* to *c*, etc. These local bending moments cause extra stresses and require additional reinforcement over and above the comparable curved section. The greater the distance between consecutive points, the greater are such local bending moments, and from a purely eco-

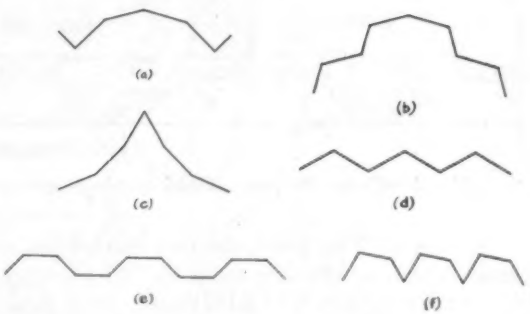


Fig. 2—Cross sections of prismatic slabs in use or considered for use

nomic point of view the engineer has to decide up to what degree the saving in formwork is offset by the extra cost of reinforcing steel.

When folded slabs whose cross-sectional shape is obtained in the way described seem to be particularly economical, there is no objection to varying this shape considerably in conjunction with requirements of usage and architecture. One reason, for instance, why a less economical shape might prove better for certain applications is the provision of a ceiling that is good acoustically. Shells, or folded slabs which follow generally the lines of shells, are well known to be bad acoustically for the ceilings of theaters and cinemas where the sound would be concentrated at certain points and others would be deficient. A shape like the one in Fig. 2(c) would be superior from an acoustic point of view.

STRENGTH AND STABILITY OF FOLDED SLABS

Fig. 2 summarizes some of the cross sections of prismatic slabs that have been used, or are being considered for use, and it is the simple zig-zag shape in Fig. 2(d) which has found the widest application; apart from its simplicity it has the great advantage that the diaphragm is reduced to a simple tie member. This tie member, together possibly with a thickening of the slabs, forms a triangle which completely stiffens the construction.

From the calculations described later, it is evident that prismatic folded slabs (which are the most frequent application) would be generally stable even if completely hinged along the lines of intersection, which are referred to as "quoins." The fact that they are usually not hinged serves to give additional stiffness which is not generally taken into consideration; in some of the more advanced applications use is made of this extra stiffness to obtain shapes that would otherwise not be possible—for instance, to provide a cantilevered plane (Fig. 5) or to produce simplified diaphragms.

Use of precast concrete

Generally, under otherwise equal conditions a folded slab will have to be thicker than a corresponding shell in order to withstand the local bending moments. However, it is possible to produce bending-resistant slabs of small thickness with ribs, and for this purpose it is often advantageous to use pre-

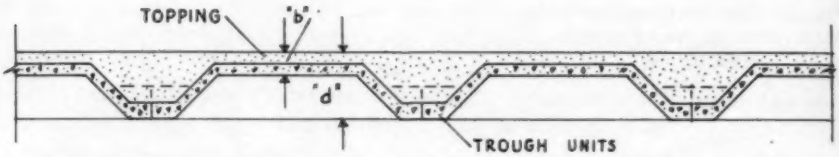


Fig. 3—Composite precast and cast-in-place construction for "shell" roof

cast concrete. This particular fact has led the author to the conviction that precast concrete, often "composite," i.e. together with cast-in-place concrete, is extremely suitable for folded slabs. Such composite construction is possible wherever the slope of any part of the roof is not greater than 25 to 30 deg, so that the top concrete can be placed without difficulty and without additional formwork. The precast concrete in these cases may consist very suitably of a number of trough units (see Fig. 3). They are placed side by side, and provide strength together with good bond to the concrete placed on top. In effect the thickness of the trough roof, together with the topping, i.e. the dimension b in Fig. 3, constitutes the thickness of the actual shell taking the direct and shear stresses which arise from the load, while the dimension d (depth of the ribs) is the criterion of the stiffness against bending.

FOLDED SLAB DESIGN

The calculations for folded slabs follow in general the lines laid down by Professor Craemer in Germany, and by A. J. Ashdown in *The Design of Prismatic Structures*. A folded slab consists of a number of slabs intersecting in straight lines (quoins). It can be assumed that each slab has a considerable stiffness in the direction of its plane while it is much more deformable in the cross direction. Any load occurring along a quoin could resolve itself in the direction of the two planes adjoining it, and be transmitted to any supports by these planes acting as beams of great depth and little width; for instance, in Fig. 1 a load acting at Quoin c would resolve itself in the direction cb and cd, and the two planes cb and cd would act as deep beams transmitting the load to whatever supports are provided.

Shear force at quoins

When this principle is applied everywhere it will be obvious that all quoins can be considered as beams, i.e. lines along which loads are supported, and it is therefore possible to assume as a first approximation that the actual slab is supported all along the quoins. Each plane receives a load from both adjoining quoins, for instance, Plane bc from Quoin b and Quoin c. These two loads may add to, or deduct from, each other. When the stresses in each plane, acting as a beam, are calculated according to the ordinary theory of elasticity, it will be found that the two planes which touch at any quoin have incompatible stresses; for instance, Plane ab might have at b a considerable compressive stress, while Plane bc had a tension stress at the same point.

Because the strain must be the same in both cases it is obviously not possible that two neighboring points should have different stresses, and it is necessary to assume a distributed shear along each quoin which equalizes these longitudinal normal stresses.

An equation can be established at every quoin for such shear force, and there are as many equations as there are quoins. In fact, these equations can be shown to have the same form as the three-moment equations for beams on as many supports, and they can be solved in exactly the same way as these. Once these shear forces, commonly called T-forces, are found, the stresses in the whole system can be established.*

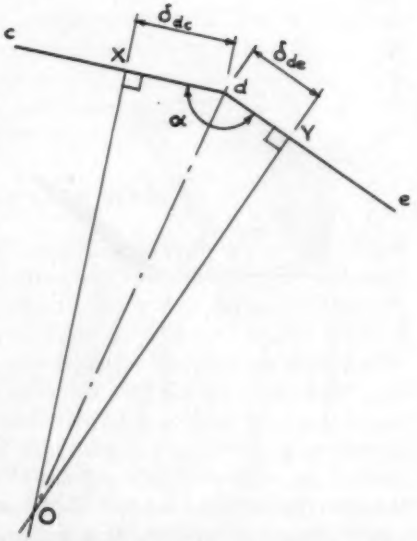


Fig. 4—Determining deformation of a quoin

Differential deformations in the slabs

Where the deflections remain of a size comparable with ordinary beam and slab construction, the first approximation described can be considered sufficient. However, it must be appreciated that folded slabs can easily change their cross section due to individual deflection, and such change in cross section may be sufficient to alter the flow of stresses completely; this will be particularly so if the angles between two consecutive planes are small. Generally speaking, the author has found that if $\sin \alpha/2 > 3a/l$ further calculations should be made (here l is the length of the prism between two supporting diaphragms, a is the average width of two consecutive panels, α is the angle between them).

When the shape of the folded slab is altered due to differential deformations, two things may occur: (a) the bending moment distribution in the slab itself spanning between the quoins may be altered, and (b) the forces into which the loads are resolved will vary. It is the second of these particularly which affects stability of the construction. The best way to deal with this is to calculate the deformation which would have arisen from the first approximation and to redraw the cross section of the construction with these deformations taken into account. It will be easily visible whether such deformations are sufficient that a new calculation is required for the altered shape; with folded slabs where the angle is very obtuse, even a third calcula-

*While the form of these equations is identical to the three-moment equation there is a considerable difference in principle. They are merely stress equations and have nothing to do with deformations; the fact that these equations are satisfied does not necessarily mean that all deformations are compatible, as in the case of the three-moment equations of continuous beams.

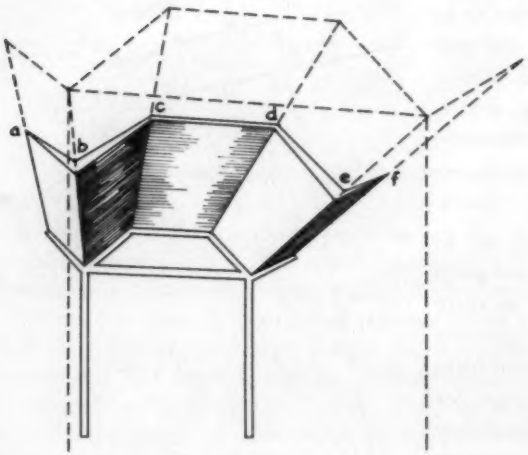


Fig. 5—Cantilevered folded plates

tion may have to be made. If the second and third approximation do not show strong convergence, this indicates that the over-all shape is unstable and should be avoided.

The interesting way of determining the deformation of a quoin is shown in Fig. 4 in greater detail. Assume that there are two consecutive planes, *cd* and *de*. Plane *cd* may have a deformation of $\delta\ dc$ in the direction dX, which can be calculated in accordance with the ordinary theory of elasticity. The same plane is also capable of moving at liberty normal to the line *cd*, as it is assumed that the stiffness in this direction is infinitesimal. Similarly, the Plane *de* would deflect by an amount $\delta\ de$ in direction dY, but could also move by an arbitrary amount normal to *de*. If lines are drawn perpendicular to *cd* at X, and perpendicular to *de* at Y, their intersection at O will be the only point to which *d* can move, which is compatible with the foregoing conditions. Therefore dO represents the deflection of Quoin *d*, and the figure shows clearly that under normal circumstances dO is considerably larger than either dX or dY, and that indeed its magnitude increases, the more obtuse the angle.

Because of the thinness of the slabs, as with shell construction, shear and diagonal tension stresses are often the most critical. They can be dealt with as in ordinary reinforced concrete, i.e. be taken by the concrete up to the limit of its capacity, and where the concrete is incapable of doing so the whole of the shear would in European practice be taken by bent up bars, generally arranged at 45 deg. (These bent-up bars are plainly visible in Fig. 9.) All compression stresses are taken by the concrete, while the total tension is taken by steel.

Cantilevers

It has previously been explained that where two planes intersect, a quoin is formed which acts like a beam. No such beam is usually available at the eaves where there is generally a wall or row of columns to take its place.

However, it is sometimes impossible to arrange for such a row of columns, and in this case the last plane must be cantilevered out as in Fig. 5. In this case quoins occur at *b*, *c*, *d*, and *e*, while points *a* and *f* are not supported. Both *ab* and *ef* cantilever out therefore, and it is good practice to make this cantilever somewhat shorter than the other planes.

POST-TENSIONED FOLDED SLABS

One way of dealing with shear stresses, as well as with longitudinal tension stresses, is post-tensioning. If a post-tensioning force acts where arranged along Quoin *a* of Fig. 1, stresses are induced in the first instance in Plane *ab*, but in the same way as before shear forces have to be introduced at Quoin *b* and also at the following quoins in order to equalize the stresses, and finally, a diagram of longitudinal stresses can be obtained over the whole cross section. It is possible, therefore, to calculate a prestressing force which is large enough to cover all tension stresses that would otherwise occur. Such prestressing forces would generally be arranged near to the line where the greatest tension occurs, and it could be of sufficient magnitude to cover all tension stresses. However, for practical reasons it is sometimes preferable to arrange post-tensioning at two or more points. The great advantage of post-tensioning in conjunction with folded slabs is the fact that the diagonal tension stresses are considerably reduced, and consequently the amount of steel necessary for shear reinforcement. When post-tensioning has been arranged the greatest diagonal stresses will not occur at 45 deg any more, but at a steeper angle, and if any reinforcement is still required it should be arranged correspondingly.

Because of the thinness of the slabs it is often not convenient to introduce stressing cables which are curved. They are best arranged straight at points where the slab can be thickened, usually along the eaves, or at other quoins.

Post-tensioning can usually be done against the weight of the construction, so that the placing of cables at the eaves is justified. However, as always when there are straight cables only, there is the possibility of tension stresses arising near the supports at the ridge of the construction, and this can be overcome by slight extra reinforcement.

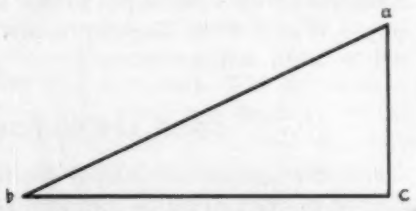


Fig. 6—Typical cross section of theater gallery

Sometimes one or all planes should be steeper than the limit previously mentioned. This can be due to architectural requirements or to the fact that a certain depth must be obtained for structural reasons, while there is not much width available to get sufficient slope. In such cases the composite construction is not very suitable because it would require rather expensive top formwork, nor would cast-in-place construction be any better. It is then possible to use only precast concrete units; they are made to act together

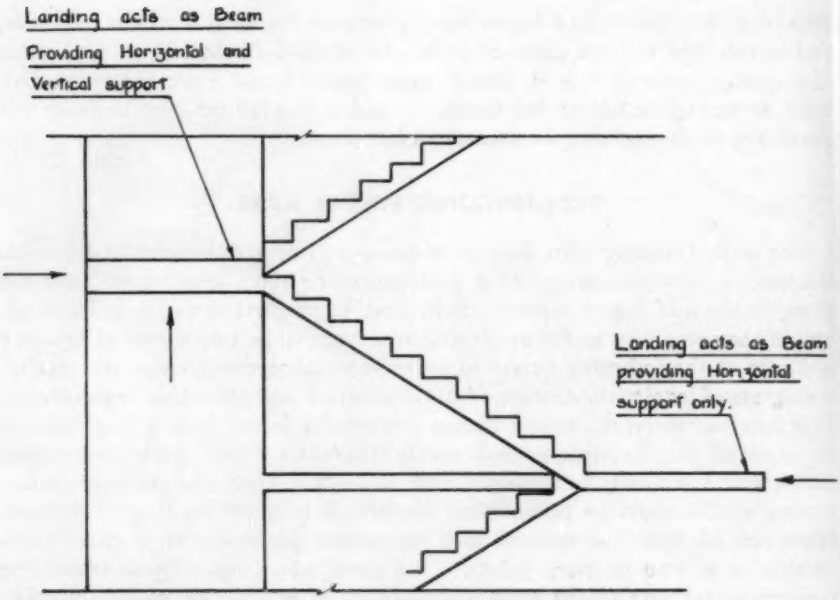


Fig. 7—Stair landing and flights may be considered as folded slab

as a beam, not by means of top concrete, but by post-tensioning only. The post-tensioning is in this case similar to the one mentioned before, but it has to be more extensive to insure that all units are compressed together efficiently so that all shear stresses can be taken. It is obviously impossible in such cases to have any diagonal bars or other shear reinforcement. The dimensions must be arranged so that the shear stress is taken by the concrete only, and so that the diagonal tension stresses are sufficiently small that they can be taken without difficulty.

OTHER APPLICATIONS OF FOLDED SLABS

Folded slabs are not entirely limited to roof construction. They can be used whenever two planes intersect, none of which contain all forces entirely. A good example is the gallery in cinemas, theaters, and similar structures. In Fig. 6 a typical cross section is shown, and if the Planes *ab* and *bc* are both solid slabs, or if one or both of them is made rigid by latticing or otherwise, the line represented in section by Point *b* can be considered to be supported by the two planes *ab* and *bc*. This is very useful because any cantilever beams become unnecessary, and if a slab can span from *a* to *b* and *b* to *c*, the whole of the interior can be kept free for air conditioning ducts or other services. A rather more complicated gallery of this type was recently carried out for the London County Council (see Fig. 13).

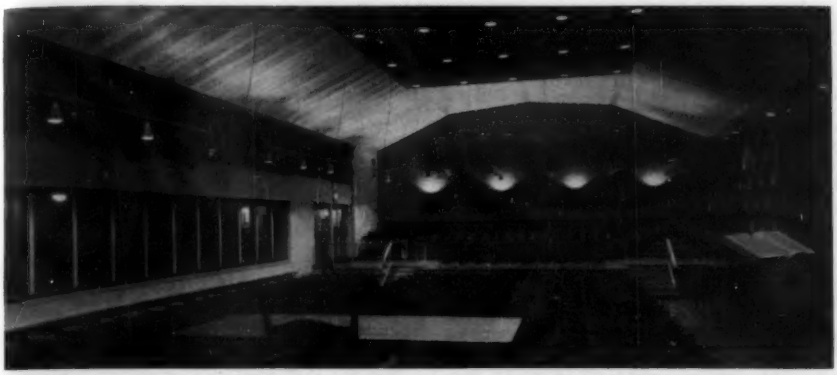


Fig. 8—Assembly hall at Thomas Linacre School, Wigan (England) roofed with folded slab, composite construction

Another application of folded slabs is found in intermediate stair landings. For architectural reasons, such landings often cannot be supported vertically at all, while a horizontal support is possible. In this case the landing, together with the flights, may be considered to be a folded slab, and a system results as shown in Fig. 7. If the main landings are fixed to the main slabs as usual and thus stiffened, this construction does not involve higher expenses than traditional construction.

EXAMPLES OF RECENT FOLDED SLAB WORK

Fig. 8 shows an assembly hall in the north of England constructed in the way described on p. 450. In the picture four rows of troughs can be seen. There were three lines of scaffolding struts, with as many temporary beams supporting the troughs when they were laid. The troughs themselves were strong enough to support the workmen and the wet concrete (Fig. 9), and of course the scaffolding and beams were removed as soon as the concrete topping and the two diaphragms (74 ft apart) had hardened. The thickness of the concrete measured through the ribs was 6 in., which was ample to get the necessary buckling stiffness. The actual total thickness of the slab was only 2 in. (1 in. of trough plus 1 in. cast-in-place concrete)—a thickness which proved sufficient to take the compressive stresses at the crown, and the average weight was equivalent to a 2½-in. solid slab. The slab was thickened slightly near the eaves in order to give cover to the concentrated reinforcement. In this case, architects Howard V. Lobb and Grenfell Baines used the pattern of the exposed troughs for the inside ceiling surface as shown in Fig. 8 on three sides of the central sounding boards.

A completely different treatment is illustrated in another assembly hall at Hatfield (architects, Easton and Robertson). In this case the roof consists of two planes only, each 25 ft long. It would have been (at that time) rather expensive to span a slab directly from eaves to ridge, so a rather more

Fig. 9—Workmen preparing reinforcement for cast-in-place portion of assembly hall roof at Wigan

complicated arrangement was chosen. As indicated in Fig. 10 giving a portion of the longitudinal section of the building, a number of prestressed planks (*A*) were laid, 5 ft 6 in. on centers, from eaves to ridge, with troughs similar to the ones previously shown covering the area between the planks. Concrete was then cast on top, and the cover was kept to 1½ in., making a total of 2½ in. with the thickness of the precast unit. In this case the shear stresses become rather large toward the end of the hall (110 ft long), and there the thickness of concrete was increased to 3½ in. including the trough. The extra weight had very little influence on the reinforcement, as it occurred only near the ends of the building.

An example of a post-tensioned folded slab is given in Fig. 11. This is an agricultural roof consisting of two planes only, 67 ft long. In this case a cast-in-place slab was arranged, and the tension at the bottom was taken by means of four post-tensioned 12-wire Freyssinet cables, accommodated in the space under the gutter. As it was not possible to curve the cables, tension stresses could occur near the ridge on either end of the building, and reinforcement was provided for this (architects, Stillman and Eastwick-Field).

As mentioned before, composite construction becomes unsuitable when the slope of a roof slab is greater than 1:3. The main roof of a church executed recently (architects, Johns Slater and Haward) has a slope of 60 deg and consists of a number of prestressed T-beams side by side with the web on the inside. In order to make all the flanges act together as a solid slab they were longitudinally post-tensioned at the eaves. Holes were left across the flanges through which the prestressing wires were pushed, the whole of the construction being temporarily supported on scaffolding. Fig. 12 shows the construction

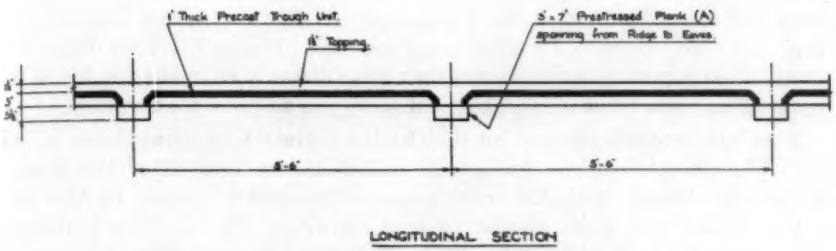


Fig. 10—Another scheme for composite folded slab

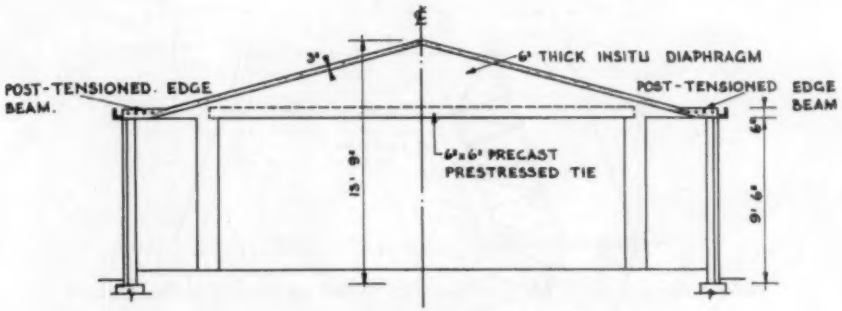


Fig. 11—Post-tensioned folded slab

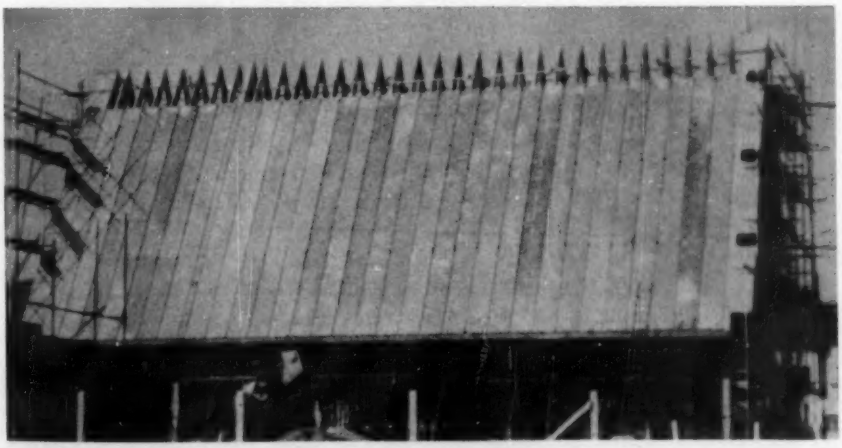
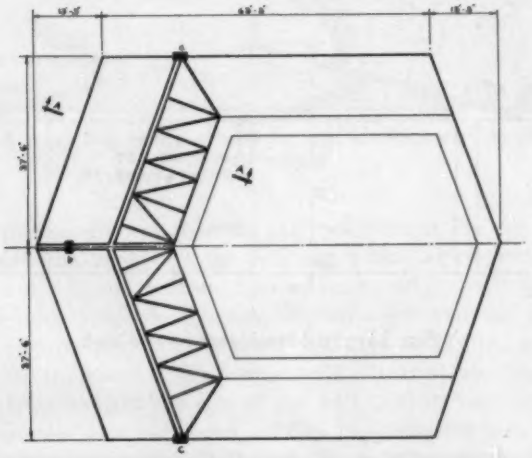


Fig. 12—Roof at 60-deg slope comprised of prestressed T-beams side by side

from outside. Notice that the flanges are cut short 6 ft from the ridge, so that any two opposite T-beams are hung against each other by their web only. This is possible since within that short distance from the top hinge the bending moment is small, and the intention was to glaze this portion.

A gallery recently planned for the London County Council is shown in Fig. 13. The auditorium was hexagonal, and a gallery that fitted the general shape was desired, with the front edge not straight in plan. In this case a folded slab was used, similar to that shown in Fig. 6. The horizontal slab was solid to comply with fire regulations. Because there were to be precast units forming steps, there was no real need for a sloping slab, and this was replaced by sloping latticed girders constructed from precast units, the ends of which had projecting bars and were cast into the edge of the



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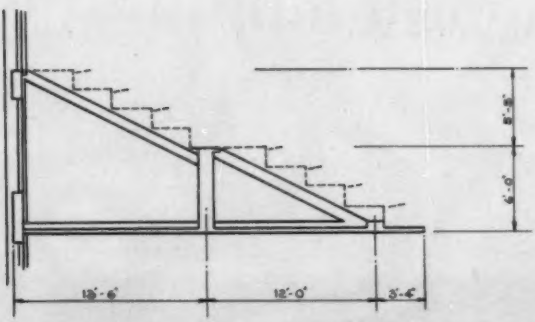


SECTION A-A

Fig. 13—Gallery for London County Council Auditorium

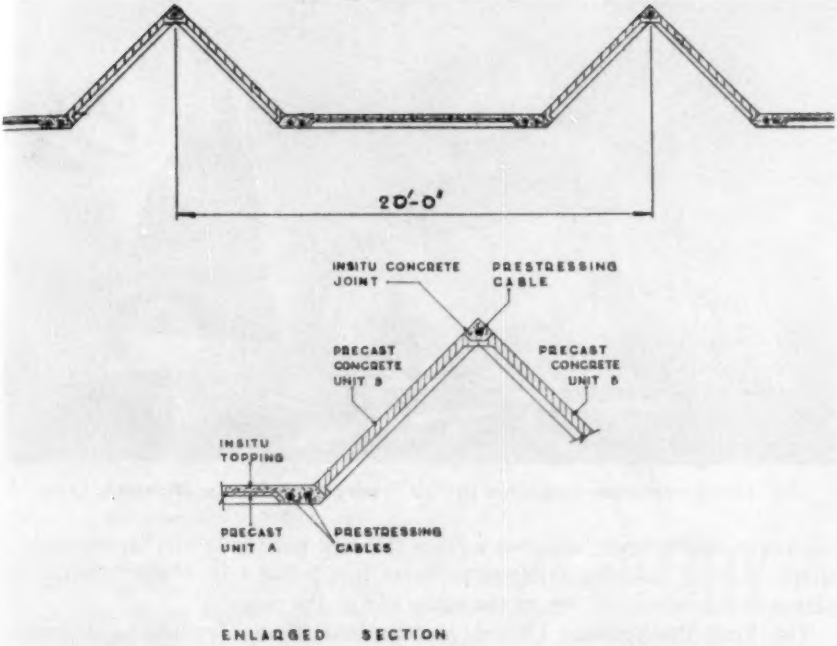


Fig. 14—Partial cross section of roof spanning 80 ft for a London school

horizontal slab. The sloping latticed girders in their turn were supported by a star beam, i.e. a beam following the shape of the building and supported at three points (*a*, *b*, *c* in Fig. 13).

Fig. 14 shows the cross section of a roof spanning 80 ft for the assembly halls and gymnasia of a high school in London (architects, Bridgwater and Shepheard). The shape was dictated by the architects, and the horizontal portions were of composite construction, while the steep portions where composite construction was not possible were made from precast units post-tensioned as in the previous example (Fig. 12). In this case a certain portion of cast-in-place concrete was arranged at the ridge and near the eaves, and post-tensioning cables introduced at these places. Strictly speaking, post-tensioned cables would not have been necessary at the ridge for overcoming the tension stresses, but were introduced in order to make sure that there was a continuous compression between the consecutive precast concrete units (*B*). Grout was introduced between these units before tensioning.

For another assembly hall now in progress, the folded slab not only forms the roof, but is carried around the side elevation so that in fact it replaces a rigid frame. Because of headroom it is carried down to a height of 6 ft 8 in. only, with the vertical portion of the frame being carried through to the bottom and made sufficiently rigid to take the bending moments caused by

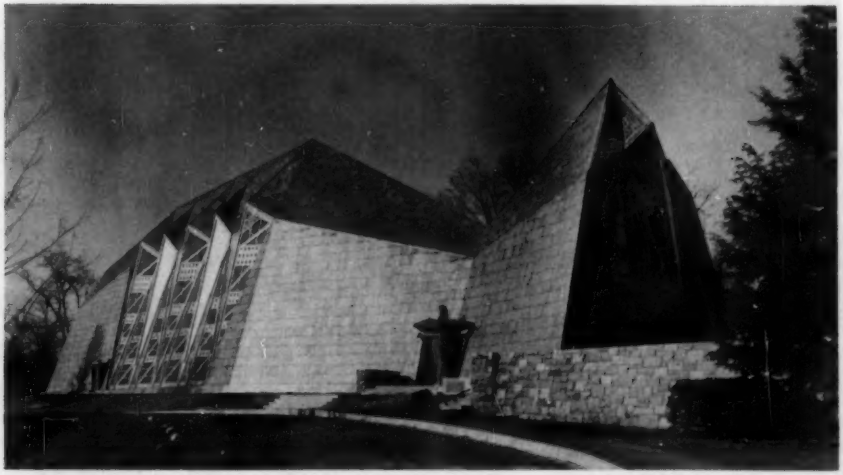
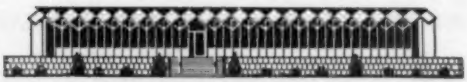


Fig. 15—Construction completed at First Presbyterian Church, Stamford, Conn.

the horizontal thrust. In cross section the roof portion of the structure is a simple zig-zag, and longitudinal stiffness has become necessary where the planes change direction, i.e. at the eaves and at the ridge.

The First Presbyterian Church at Stamford, Conn. (architects, Harrison and Abramovitz) is built on a similar principle, and represents in fact an entire folded slab construction extending over the sloping external walls as well as the roof (Fig. 15). The center portion and the west gable were "triangulated" in order to introduce a structure that is completely self-supporting without any bending stresses being transmitted from one portion to another. Each plane is one precast unit 8 in. thick, and connecting steel has been incorporated in the cast-in-place joints between the units. In all the triangular units which form the center portion of the wall large openings were introduced to carry stained glass windows. The remaining portion, which has no stained glass windows, is constructed on the same principle, but with quadrangular precast slabs.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.



Title No. 55-30

R/C core main structural element in 22-STORY OFFICE TOWER*

By OTTO SAFIR†

Because of its narrow width and the required resistance against wind forces and earthquakes, the main structural element of the tower is formed by the reinforced concrete core which houses the building services, i.e., elevators, staircases, and lavatories. This cellular core resists all horizontal forces and carries the greater part of the vertical loads.

The office section surrounding this core is a composite of structural steel and reinforced concrete to reduce the column sections to a minimum and to achieve rapid construction. The method of analysis of the core is described, as well as the construction procedure which resulted in the construction of one complete floor in 7 working days.

■ The office tower is the main element of the building complex which forms the head office for the British Columbia Electric Co., Ltd., in Vancouver. This tower is completely separated structurally from the adjacent lower blocks, and its design presented some interesting features because of its narrow width and other planning requirements.

The tower (Fig. 1) consists of a subbasement, basement, ground floor with part mezzanine, 21 typical upper floors, and a three level penthouse. It rises to a total height of 319 ft above foundation level, or 287 ft above entrance level at ground floor. In plan (Fig. 2) the tower is 200 ft long and has a width varying from 45 ft at the ends to 72 ft in the center. The shape

*Presented at the ACI 54th annual convention, Chicago, Ill., Feb. 26, 1958. Title No. 55-30 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 4, Oct. 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Jan. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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of the tower core was determined by reasons of strength and deformations under wind loads; the over-all shape of the building plan was the result of architectural considerations, mainly daylighting, i.e., no desk was to be more than 15 ft from an exterior wall and window. Width of the office space from the interior wall to the windows is 20 ft 6 in. On most floors a 4 ft wide corridor reduces the clear office width to approximately 16 ft.

STRUCTURAL DESIGN CONSIDERATIONS

The following considerations influenced the design of the structure.

The over-all height of the building was limited by city by-laws and to accommodate the maximum number of floors within this height, the floor thickness had to be kept to a minimum. With the structural system chosen it was possible to achieve a clear ceiling height of 9 ft with a floor to floor height of 10 ft 9 in., and accommodate the equivalent of 23 stories within the limiting height. To achieve this small over-all floor thickness of an air conditioned building (structural thickness is 16 in.—5 in. for floor finish and suspended luminous ceiling), it was essential to integrate fully the structure with the mechanical and electrical services.

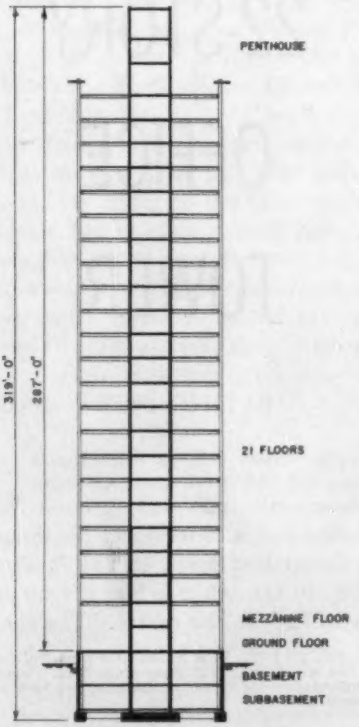

Fig. 1—Cross section through tower

Comfort of the occupants working in the building was considered essential and dictated that the horizontal movements of the building under wind forces be an absolute minimum. The building is designed to withstand earthquake forces and forces from winds up to 90 mph which, in accordance with the Canadian National Building Code, cause horizontal forces of up to 40 psf of wall area at the top of the building. Because of the narrow shape of the plan this limitation of the horizontal movements imposed particular structural problems and various systems were investigated before the final solution was evolved. In particular, systems employing multistory rigid frames had to be rejected as they required considerably greater structural depth of the floors and resulted, therefore, in a reduction of the total number of floors possible within the given height.

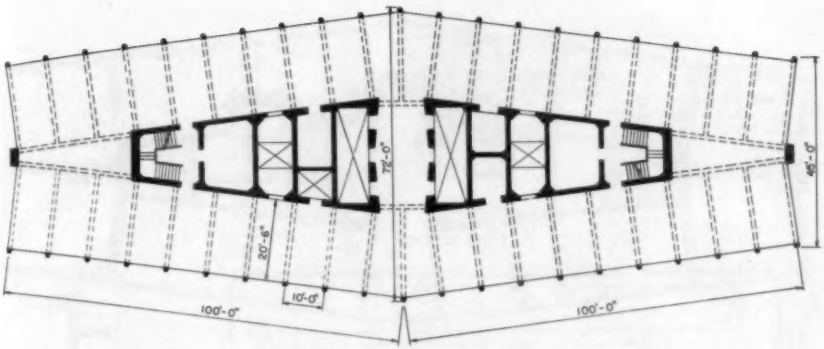

Fig. 2—Typical floor plan

Speed of construction was also an important consideration in selecting the structural system. Shoring and re-shoring of floors below those on which structural work was in progress was avoided so as to allow finishing operations to follow close behind the structural work.

STRUCTURAL DESIGN DETAILS

The structure as developed comprises, as the main element, a reinforced concrete core constructed similar to a grain elevator with intermediate floor slabs. This core houses all the essential services in the building, i.e., elevators, mail conveyors, ventilation and other service ducts, staircases, and lavatories. The interior walls of this reinforced concrete core are parallel to the exterior walls of the building, from which they have a distance of 20 ft 6 in. The reinforced concrete core resists all horizontal forces caused by wind or earthquakes, and it carries the greater part of the vertical loads of the building.

The floor construction of this 20 ft 6 in. wide strip, which is the actual office area, is carried on the reinforced concrete core, and on the outside on columns spaced 10 ft on centers. It consists of beams spanning between the columns and the core walls with reinforced concrete floor slabs spanning 10 ft from beam to beam. The floor beams spanning between the core walls and the exterior columns (20 ft 6 in.) are partially fixed in the core walls, and this arrangement reduces the loads in the exterior columns (Fig. 3).

These columns project to the outside of the building, giving a flush interior face to all offices and acting as main grid lines for the curtain wall cladding of the building. The columns are of uniform section in all typical floors to facilitate the curtain wall construction. The columns are constructed as composite sections comprising a structural steel core encased in reinforced concrete. On the lower floors this structural steel core consists of two steel plates welded together to form a T, and on the upper floors the steel core com-

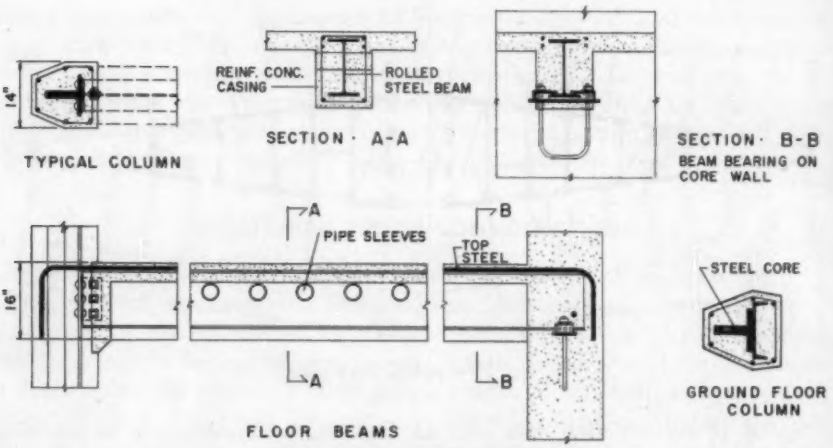

Fig. 3—Structural details

prises two unequal angles back to back. Although it was not necessary for strength reasons, the steel core was continued with a minimum section in the uppermost floors for uniformity of erection procedures.

The beams carrying the floor of the office section are also composite, consisting of a rolled steel beam encased in reinforced concrete. The steel beams are connected to the structural steel cores of the columns and are supported on bearing plates on the walls of the reinforced concrete building core. The steel sections carry, as simply supported beams, the dead weight of the concrete, i.e., the formwork for the concrete floor slabs and beam encasings was suspended from the steel beams. Additional round steel reinforcement resists the fixing moments of the beams at the columns and at the core walls, and all loads other than the dead weight of the reinforced concrete are carried on the partially fixed composite section. This method of floor construction avoided the need for any shoring and re-shoring of floor slabs.

BUILDING CORE

The reinforced concrete core of the building is designed as a cantilever structure above the ground floor slab, resisting all horizontal forces due to earthquake or due to wind forces of up to 40 psf at the top of the building. The full width of the building is used as a base below the ground floor. The basement and subbasement are a cellular structure with crosswalls at approximately 30 ft on centers (Fig. 4), out of which the reinforced concrete core rises at ground floor level. The foundation below the subbasement consists of a pad under the actual core with strip foundations under the exterior and under the crosswalls of the basement. The average bearing pressure of the foundations is 14 kips per sq ft carried on a medium hard

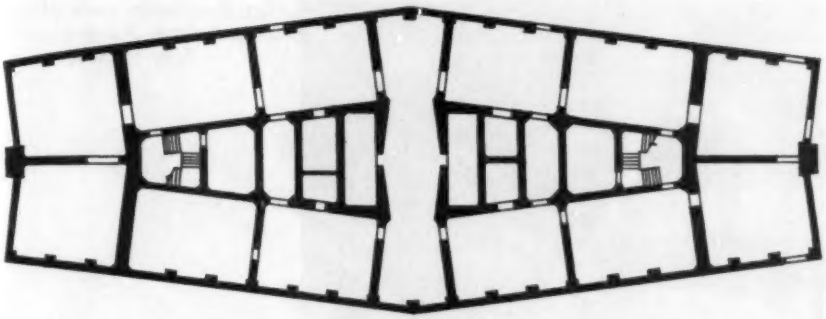

Fig. 4—Basement plan

sandstone. The crosswalls of the basement and subbasement withstand the unbalanced forces caused by the horizontal loadings and by any uneven settlement of the foundations.

The reinforced concrete core is essentially a gravity structure. The thickness of its walls varies with the height; a minimum wall thickness of 8 in. was used at the upper floors and this was increased to 15 in. on the lower floors and 16 in. on the basement and ground floor.

An analysis of the core structure showed maximum tensile stresses of approximately 200 psi for a combination of minimum vertical loads with maximum wind or earthquake moments. These are tensile stresses taken

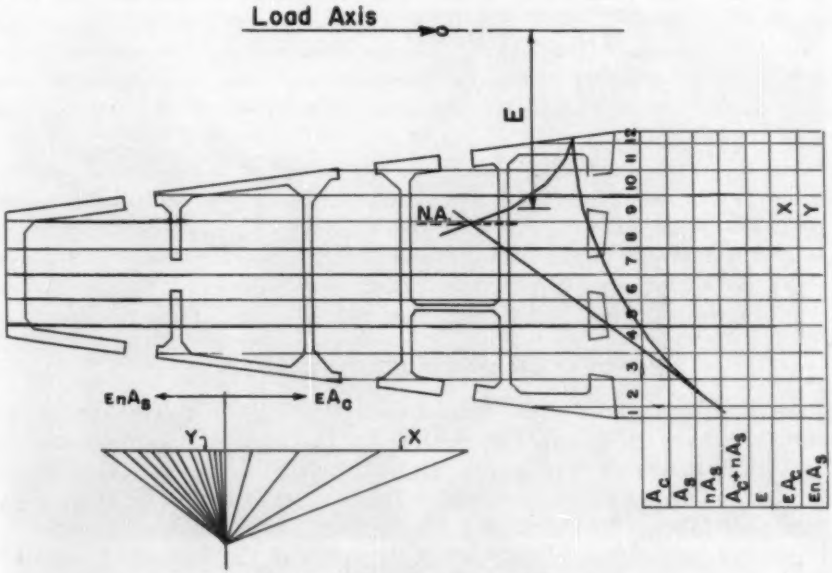

Fig. 5—Graphic analysis of building core

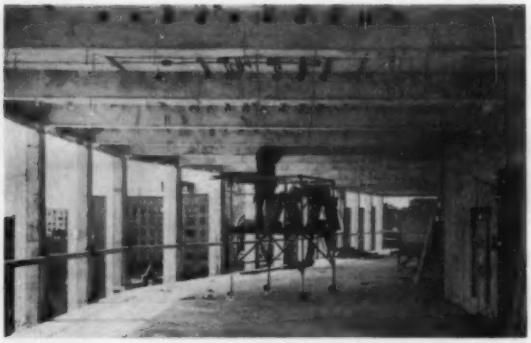

Fig. 6—Interior view of completed floor showing underfloor conduits

on the uncracked concrete section. This tensile stress is well below the accepted tensile strength of the 4000-psi concrete employed, but it does not provide an indication of the true factor of safety of this main structural element and neglects effect of construction joints on bond.

The stresses in the core were, therefore, also examined for "ultimate" wind loads on the basis of a cracked section. An ultimate load factor of 4.00 was used, combining such "ultimate" wind moments with the minimum dead loads. Fig. 5 shows the principle of the graphic analysis used to determine the neutral axis of the "cracked" reinforced concrete section based on the method developed by Spangenberg. [The area of the section, both concrete (A_c) and steel (A_s), transformed to (nA_s) is divided into small strips (1 to 12). The products of these areas multiplied by their distance from the load axis are considered as weights whose center of gravity is the neutral axis of the section for the loading. The funicular polygon construction shown in Fig. 5 determines the neutral axis taking into account the concrete sections only on the load side of the neutral axis. The stresses in the concrete and steel are then determined.]

On this basis "ultimate" maximum fiber stresses in the concrete were 4000 psi at ground floor level and tensile stresses in the steel were 65,000 psi. The horizontal movement of the building at roof level under forces caused by a 90 mph wind was $2\frac{1}{2}$ in., and this figure was considered permissible. Since the building was completed no damage to partitions or other installations due to lateral movements has been observed.

INTEGRATION OF STRUCTURE WITH SERVICES

As mentioned earlier, the reinforced concrete core accommodates the vertical runs of all essential services in the building. The air conditioning ducts in the actual office section are accommodated within the depths of the beams below the floor slab, and in larger supply ducts running along the core walls above the corridors. Electrical underfloor services are carried in conduit located immediately below the soffits of the floor slabs accessible from the floor above and also accessible from below through the luminous

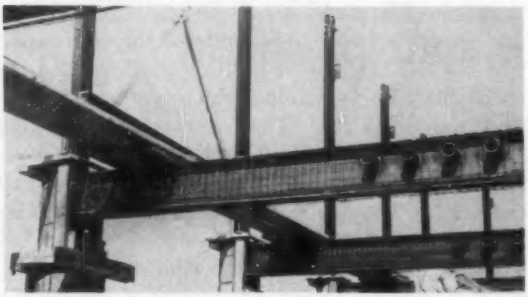

Fig. 7—Erection of steel columns and floor beams

ceiling. Provision was made in the floor beams for the great number of electrical services, the steel beams were fitted with pipe sleeves for the passage of the conduits (Fig. 6).

CONSTRUCTION PROCEDURE

After construction of the basements in the usual way, erection of the superstructure of the tower commenced on a regular schedule. Forms for the core walls and floor slabs of the core were erected, reinforcement was placed, and these walls were concreted to the underside of the steel beams.

The structural steelwork was erected after removal of the exterior wall forms the following day (Fig. 7). The structural steel columns were spliced at every second floor and adequately braced with temporary bracings. The

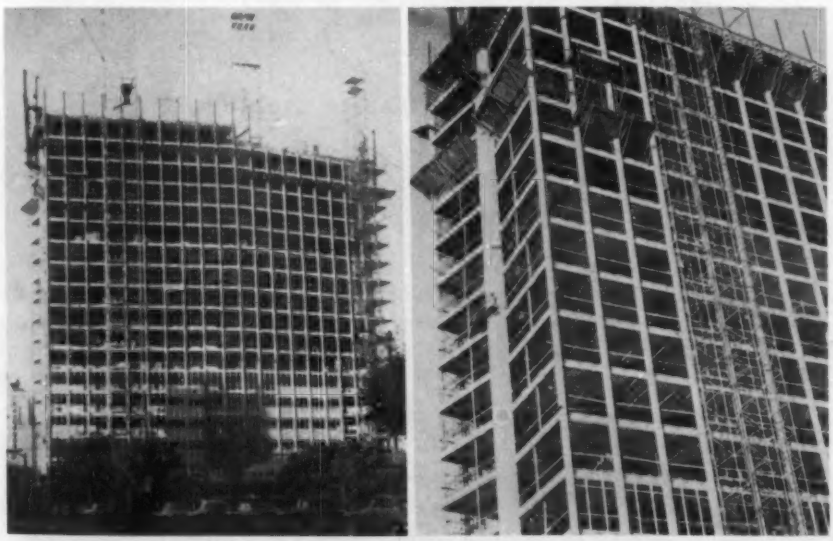

Fig. 8—Two views of tower under construction. Curtain walls are being installed on lower floors. Note the end column in view at right

rolled steel beams were connected to the steel columns by high tensile bolts and supported on the concrete of the core walls with steel bearing plates adjustable for height and position and clamped to the steel bearing plates with small bulb angle clamps which were welded when erection of one-half the floor was complete. The formwork for the column casings was then placed and formwork for the floor beams and floor slabs was suspended from the rolled steel beams. After placing of the reinforcing steel, concrete was placed. One-half of a tower floor was constructed in this fashion at a time, and a completion cycle of 7 working days for a typical tower floor was achieved. Fig. 8 shows the structure in the course of erection; the structural steel columns with bracing are visible at the top of the building, and fixing of the curtain wall has commenced on the lower floors.

Excavation for the building commenced in July, 1955, and the floor slab of the first floor was cast in January, 1956. The last floor slab was cast in September, 1956, and the building was completed and occupied by the owners in February, 1957.

ACKNOWLEDGMENT

Architects for the building were Thompson, Berwick and Pratt of Vancouver. Structural design was by the author. John Laing & Son (Canada), Ltd., were the general contractors.

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research and development

Title No. 55-31

PRECAST CONCRETE GIRDERS reinforced with HIGH STRENGTH DEFORMED BARS*

By J. R. GASTON and EIVIND HOGNESTAD†

Two 0.38-scale model roof girders were tested to develop an unusual type of precast concrete building frame. The roof girder selected departs from customary practice primarily by its slender and graceful T cross section, by its high strength longitudinal tension reinforcement, and by its inclined stirrup reinforcement. Structural design was based on the ultimate strength design procedure given in the appendix of the 1956 ACI Building Code, with some departures justified by the model girder test results. Twenty 58-ft girders were later manufactured for two laboratory buildings.

UNUSUAL PRECAST BUILDING FRAME

In early design stages of two new laboratory buildings at the Portland Cement Association Laboratories, it was decided that recent developments in structural concrete would be utilized to a greater extent than is at present customary in building construction. Functional requirements of the two buildings, the Structural Laboratory and the Fire Research Center, called for an open crane-bay superstructure about 58 ft wide and 40 ft high from first floor to roof level. Furthermore, special requirements related to the use of the buildings as laboratories called for extremely heavy box-like foundations in both cases.

The superstructures were ideal for precast concrete construction since considerable repetition of identical structural units was possible within each

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structure as well as between the two structures. Preliminary designs therefore turned to a system of precast columns, wall panels, roof girders, roof slabs, spandrel beams, and crane girders.

The ultimate strength design method given in the appendix of the 1956 ACI Building Code (ACI 318-56)* was used in the structural design. However, it was decided to exceed moderately the 60,000 psi maximum value for stress in reinforcement at ultimate strength given by Section A603(e). Design of some details, such as roof girder web reinforcement, also departed somewhat from the limits and principles of ACI 318-56.

Thus, the experimental investigation reported herein was undertaken to develop and to test an unusual type of precast concrete building frame. The roof girder selected departs from customary practice by its slender T-shaped cross section, by its high strength longitudinal reinforcement, and by its inclined stirrup reinforcement.

Two model girders were tested. The purpose of the first test was to determine the general behavior of the girder from zero load to ultimate strength, with particular attention given to flexural cracking and performance at service load. In the second girder test, attention was focused on adequacy of web reinforcement, action of face bars provided to reduce the width of cracks in the web, and the strength and performance of the girder-column connection.

TEST GIRDERS

Prototype frames

The frame system and the roof girder shape were selected as a result of suggestions originating with Ulf Bjuggren and Torsten Lundin of Stockholm, Sweden. Columns 2 ft square and about 38 ft tall are fixed to the foundation. Crane girders and spandrel beams span 22 ft longitudinally between columns, and roof girders span 58 ft transversely between pairs of columns. The roof girders are connected to the columns in a manner designed to permit rotation without development of negative frame corner moments. However, the connection permits a horizontal force to be transmitted in each frame through the girder from one column top to another. Thereby both columns of a frame may be designed to "share" sidesway moments due to wind and crane loads. The maximum compression so transmitted through the girder is less than 5 percent of the internal compression due to bending of the girder.

The roof girders selected have a T-shaped cross section 4 ft deep at midspan and 3 ft deep at the ends. Structural frame design was carried out for a design minimum concrete strength of 5000 psi and a design minimum yield point for the main longitudinal tension reinforcement of 70,000 psi. A longitudinal tension reinforcement of eight #9 bars with a total area of 8.0 sq in. satisfied design requirements.

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, May 1956, *Proc.* V. 52, pp. 913-986. Further reference to Code is given by section only.

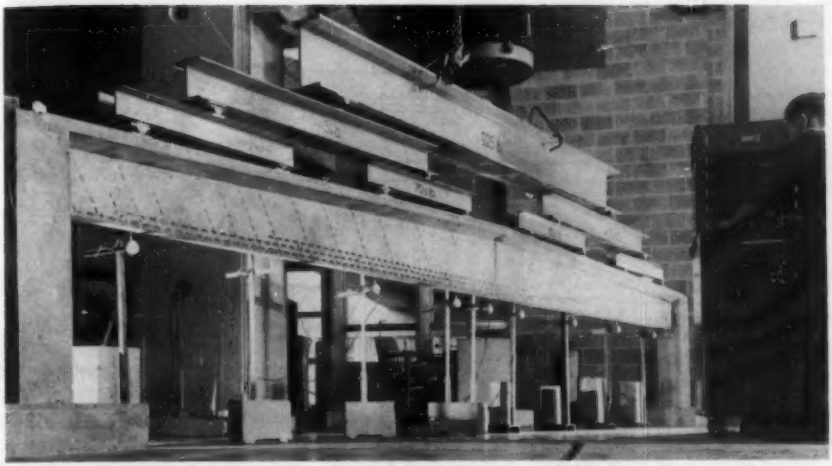


Fig. 1—Testing arrangement for Girder 1

Test specimens

A 0.38-scale model of the prototype was the largest specimen that could be conveniently handled in the laboratory. Fig. 1 and 2 show over-all views of the testing arrangement for the two test girders, and their dimensions are given in Fig. 3. The total depth of the test girders was 0.38 × 4 ft = 18¼ in. at midspan, and 0.38 × 3 ft = 13½ in. at the ends. To simulate the roof loading of the prototype, the test girders were loaded at eight points as shown in Fig. 3.

The columns of the prototype frames were designed in accord with the appendix and other applicable provisions of ACI 318-56. The column design procedures involved, therefore, are substantiated by numerous previous tests. It was necessary in this investigation, however, to develop a satisfactory connection between girders and columns. To accomplish

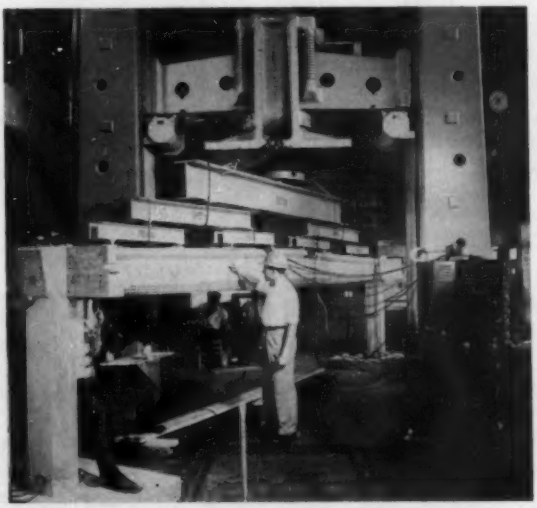


Fig. 2—Testing arrangement for Girder 2

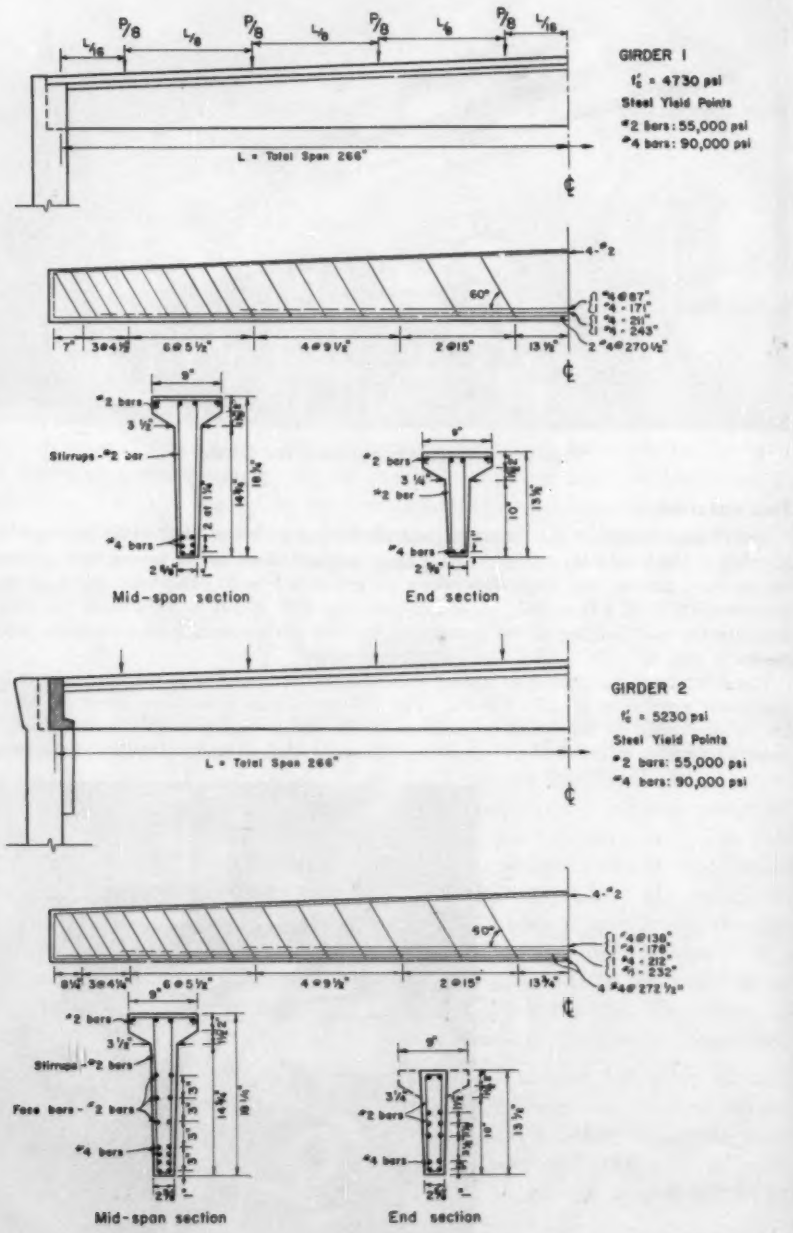


Fig. 3—Test girder details

this, the test girders were supported on column stubs as shown by Fig. 1 to 4. For Girder 1, the connection was merely a pocket in the column head into which the girder was seated. The girder rested on a steel bearing plate, and the space between girder and column pocket was grouted. A bituminous paint was applied to break bond and thus prevent continuity between girder and column. Caulking compound filled the space under the girder in front of the bearing plate. The connection for Girder 2 was a 0.38-scale model of the one selected for use in the prototype. Stubs of the spandrel beams were provided, and the girder flange was discontinued at the face of the spandrel beams. A ¼-in. bolt was used to secure the column to the girder. This bolt serves erection purposes only. It is inactive for all loading cases considered in the prototype design.

Materials and fabrication

Design minimum concrete strength for the prototype was 5000 psi. The concrete mixture used in the laboratory, 1:2.5:1.7 by weight, was proportioned to produce this strength at an age of 1 week. The cement used was a laboratory blend of four brands of Type I portland cement. The cement content was 7½ bags per cu yd, w/c by weight was 0.45, coarse aggregate was a gravel of ⅜ in. maximum size, and slump was 3 in. Girder 1 was tested at an age of 7 days when the average strength of 6 x 12-in. test cylinders made and cured with the girder specimen was 4730 psi. Girder 2 was tested at 8 days, and the cylinder strength was 5230 psi.

Main tension reinforcement with a minimum yield point of 70,000 psi was selected for the prototype girders in which the ultimate strength was controlled by tension. However, to test the strength of the compression zone and the web, flexural failure was delayed by using main tension reinforcement with an average yield point of 90,000 psi in the model girders. The #4 deformed bars (ASTM A 305) were rolled from alloy steel billets, AISI No. 9261.

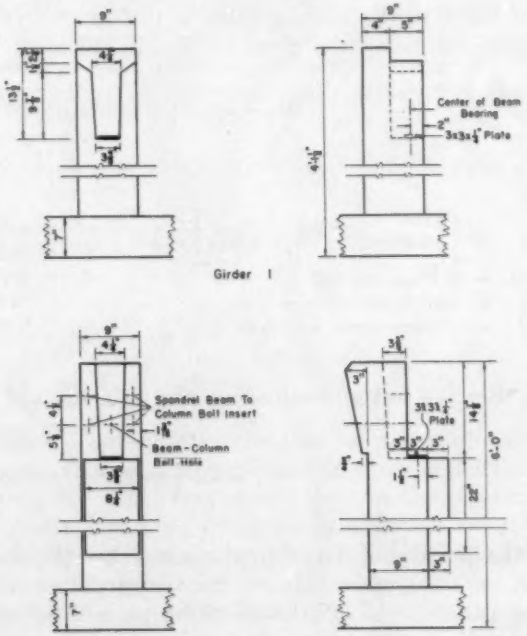


Fig. 4—Column stub details

The physical properties of these bars are given in Fig. 5. The compression reinforcement, stirrups, and face bars of the test girders were #2 deformed bars having an area of 0.045 sq in. and a yield point of 55,000 psi.

The girders were cast in plastic coated plywood forms with close adherence to the dimensions given in Fig. 3. The fabrication and placing of reinforcement cages was carried out to tolerances of less than $\frac{1}{4}$ in. Concrete was placed in the test girders and companion cylinders by internal vibration. Following 3 days of moist curing, all specimens were stored dry in the laboratory until they were tested. It was noted that the taper of the web permitted the girders to be lifted out of the forms without dismantling the forms. This could simplify form design and mass production of the type of girder involved.

Design of test girders

The main tension reinforcement of Girder 1 consisted of six #4 bars with a total area of 1.20 sq in. This represents closely the area corresponding to the prototype, namely, $8 \times 0.38^2 = 1.16$ sq in. To design web reinforcement for the test girders, and to study their service load behavior, it was necessary to calculate a service load for the test girders corresponding to that of the prototype. For this purpose, compression reinforcement and the small axial force were neglected and the prototype's 70,000 psi yield point and 5000 psi concrete strength were assumed. Flexural strength is controlled by the section at the load points nearest midspan, for which the effective depth $d = 15.5$ in. By ACI 318-56, Section A607(a), the depth to the neutral axis at ultimate strength $= 1.30\ qd = 1.30 \times 0.12 \times 15.5 = 2.4$ in. This depth is less than the average flange depth of $t = 3.0$ in., and $q = 0.12$ is less than the limiting value at 0.40 given by Section A605(c), so that the ultimate moment is given by Section A605(b):

$$M_u = bd^2 f_c' q\,(1 - 0.59q) = 1210 \text{ kip-in.} \qquad (1)$$

in which

$t = \frac{1}{9}\left[3.5^2 + \frac{9 - 3.5}{2}(2 + 3.5)\right] = 3$ in.

M_u = ultimate moment
b = total flange width = 9 in.
d = effective depth = 15.5 in.
f_c' = cylinder strength = 5000 psi
q = $A_s f_y / bd f_c' = 0.12$
A_s = area of tension reinforcement = 1.2 sq in.
f_y = tension steel yield point = 70,000 psi

For the test girder loading, the ultimate load is:

$$P_u = M_u \frac{8}{L} = 1210 \frac{8}{266} = 36.4 \text{ kips}$$

The live load of the prototype is less than the dead load, so that a load factor $K = 1.8$ applies. Hence, the test girder service load including the weight of the girder is $36.4/1.8 = 20.2$ kips, and the service load shear, V, at each beam end is 10.1 kips.

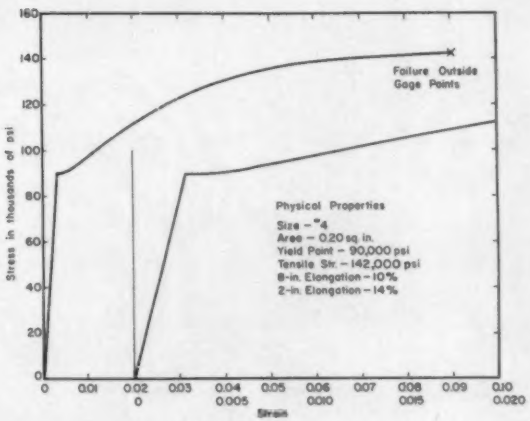


Fig. 5—Physical properties of AISI No. 9261 deformed bars

Shearing stress at the girder end is:

$$v = \frac{V}{bjd} = \frac{10{,}100}{3 \times 0.9 \times 12.75} = 293 \text{ psi} \quad \ldots\ldots(2)$$

which exceeds the 240-psi ceiling given by Section 803(c) for beams with vertical stirrups used alone as web reinforcement. Stirrups inclined 60 deg were chosen as web reinforcement for the prototype. Since high strength longitudinal reinforcement cannot be welded in the field, the stirrups were only tied to the main reinforcement with soft wire, although Section 802(a) calls for inclined stirrups to be "welded or otherwise rigidly attached to the longitudinal steel."

The stirrups were designed by Section 804(d) of the ACI Code:

$$A_v = \frac{V' s}{f_v jd (\sin \alpha + \cos \alpha)} \quad \ldots\ldots(3)$$

in which

A_v = stirrup area = $2 \times 0.045 = 0.09$ sq in.
s = horizontal stirrup spacing
f_v = tensile unit stress = 20,000 psi
jd = $0.9d$
α = stirrup inclination = 60 deg

By Section 801(d), the shear, V', carried at service load by web reinforcement should be taken as the excess over a 90 psi shearing stress permitted for the concrete of an unreinforced web. It was felt that, in the present case of an unusually slender girder with high strength materials, the resulting stirrup reinforcement would be too light in the midspan region. It was chosen, therefore, to design the stirrups throughout the girder span for a value of V' equal to 2/3 of the total shear present at a section. The stirrup reinforcement designed in this manner is shown in Fig. 3.

Girder 2 had web reinforcement identical to that of Girder 1. The longitudinal main reinforcement was increased to eight #4 bars, thus still further delaying flexural failure by exceeding both the yield point and the steel percentage of the prototype. This was done to ascertain the strength of the web reinforcement. Face bars were added to one-half of the girder to determine their effect on flexural cracking.

TEST RESULTS

The general behavior of both test girders was entirely satisfactory throughout the test. In both cases ultimate strength was governed by yielding of the main tension reinforcement below one of the two load points located $L/16$ from midspan. Girder 2 failed in the half without face bars.

Ultimate flexural strength

The ultimate loads P_{test}, including dead weight of girder and loading frame, are given in Table 1 together with other pertinent strength data. Using 90,000 psi longitudinal reinforcement, the average flange depth of 3.0 in. is less than the depth to the neutral axis given by 1.30 qd [Section A607(a)]. Accordingly, the ultimate moment is given by Section A607(b) of the ACI Code:

$$M_u = (A_s - A_{sf}) f_y d \left[1 - 0.59 (q_w - q_f)\right] + A_{sf} f_y (d - 0.5t) \quad \ldots\ldots\ldots (4)$$

in which for Girders 1 and 2, respectively:

A_s = area of tension reinforcement = 1.20 and 1.60 sq in.
A_{sf} = 0.85 $(b - b')$ t f_c'/f_y = 0.804 and 0.890 sq in.
f_y = yield point of tension reinforcement = 90,000 psi
b = flange width = 9 in.
b' = average width of web = 3 in.
t = average flange thickness = 3 in.
f_c' = cylinder strength = 4730 and 5230 psi
d = effective depth = 15.5 and 15.25 in.
q_w = $A_s f_y / b' d f_c'$ = 0.491 and 0.602
q_f = $A_{sf} f_y / b' d f_c'$ = 0.329 and 0.336

The corresponding calculated ultimate loads, P_{calc}, are given in Table 1. It may be noted that the 60,000 psi yield point limit given by Section A603(e) was exceeded and that compression reinforcement was neglected. The values of $q_w - q_f$ given in Table 1 are substantially below the maximum value of 0.40 given by Section A607(c), so that Eq. (4) is applicable.

Ultimate flexural strength was also calculated by more refined methods using a curved stress-strain relationship for the concrete and using the actual girder cross section. For an assumed trapezoidal steel stress-strain relationship, calculated ultimate loads were obtained within a few percent of those given in Table 1 for Eq. (4). Using the actual steel stress-strain curve of Fig. 5, however, calculations give a steel strain of about 0.01 at ultimate strength, which corresponds to a steel stress by strain hardening of 100,000 psi. The resulting calculated ultimate loads are in close agreement with the observed ultimate loads, P_{test}, given in Table 1.

TABLE 1—GIRDER FLEXURAL STRENGTH

Girder No.	f_c', psi	A_s, sq in.	d,* in.	q†	$1.3\omega d$, in.	$q_e - q_f$	P_{test}, kips	P_{calc}, kips	$\frac{P_{test}}{P_{calc}}$
1	4730	1.20	15.5	0.164	3.3	0.162	50.4	45.4	1.11
2	5230	1.60	15.25	0.301	4.0	0.266	67.2	57.6	1.16

*At load points $L/16$ from midspan.
†Based on full 9-in. flange width.

It may be concluded that, even in the present case of an unusually slender girder with high strength materials, a reliable and slightly conservative ultimate flexural strength design value may be obtained by Eq. (1) or (4), whichever is applicable.

Girder deflection

Deflections of Girder 1 were measured by dial gages (Fig. 1), and those of Girder 2 were obtained utilizing scales fastened to the girder and read with a surveyor's level. Midspan deflections are given in Fig. 6a as a function of the total load. It is seen that Girder 1, which had tension reinforcement corresponding to that of the prototype, had a deflection of about 1 in. at full service load. This corresponds to $1/0.38 = 2.6$ in. for the prototype. The prototype service load consists of about 27 percent live load and 73 percent dead load. Assuming that dead load deflections will be doubled by the effect of creep, the total prototype deflection is $2.6\ (2 \times 0.73 + 0.27) = 4.5$ in. The prototype girders were therefore built with a camber of 4.5 in.

Girder deflection was calculated using the moment of inertia of the gross concrete section, considering the effect of the variable girder depth, neglecting reinforcement, and using a modulus of elasticity equal to $1000\ f_c'$. A load-deflection line obtained for 5000-psi concrete strength is plotted in Fig. 6a. It is seen that, for the unusual girder involved, the measured deflection at service loads exceeds the calculated value.

By a second method, deflection was calculated on the basis of the following assumptions: (1) concrete and steel stress are proportional to strain; (2) the concrete section is cracked, and no tensile stress is carried by the concrete below the neutral axis; and (3) the modular ratio is $n = 30{,}000/f_c'$ as given by Section 305. The load-deflection curves obtained by this cracked-section method are also plotted in Fig. 6a. At low loads, the actual deflections were less than those so computed, because the girders were actually not cracked. At loads near the yield loads, actual deflections exceeded those calculated, because concrete and steel were approaching plastic action. In the region of service loads, however, deflection computed by the cracked-section method is in close agreement with measured deflections.

Flexural strain

Strain measurements were made on Girder 2 only. Flexural strain was measured near the two central load points at sections $22\frac{5}{8}$ in. from midspan.

At each section, strain in the compression flange was measured by five 6-in. electric strain gages attached to the concrete surface. A 10-in. Whittemore mechanical gage was used to measure strain at four levels in the web. Finally, strains for all four steel layers were observed by electric gages attached to the reinforcement, waterproofed, and embedded in the concrete.

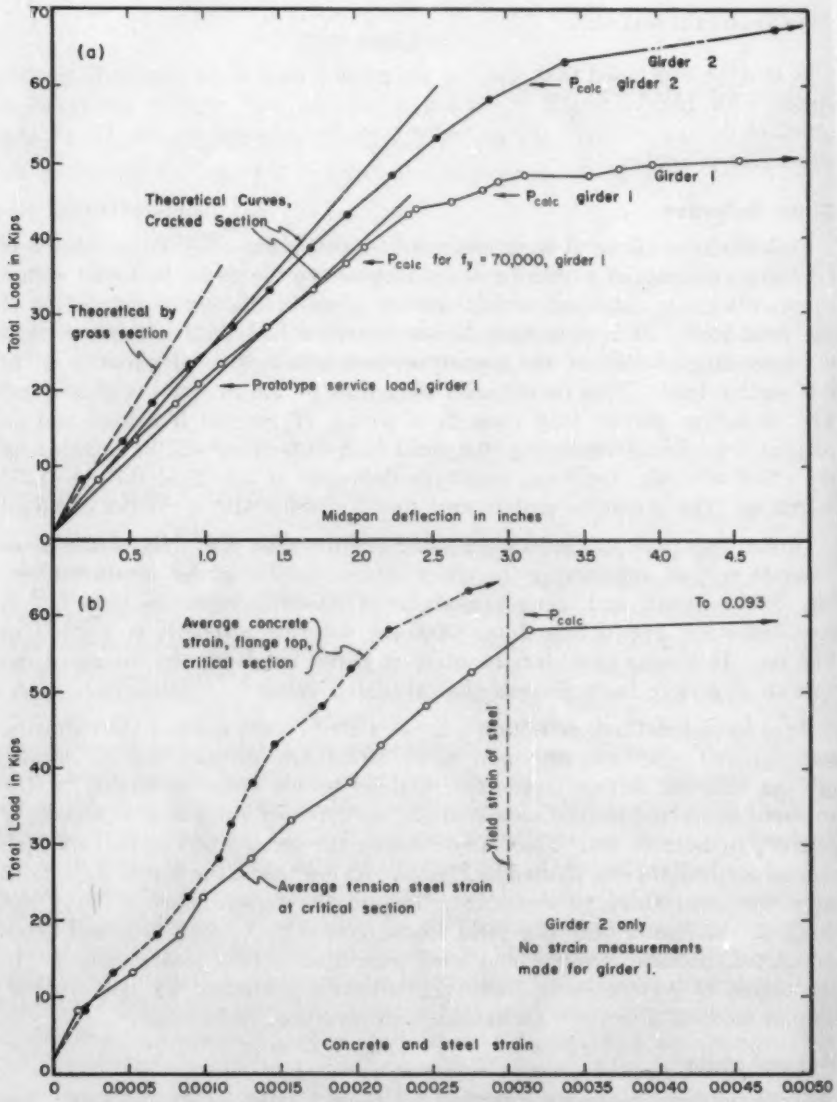


Fig. 6—Deflection and strain data

The distribution of strain across both sections was linear from the first load increment to failure. This substantiates numerous previous observations justifying the design assumption that plane sections normal to the axis remain plane after bending. Average concrete and steel strain are given in Fig. 6b as a function of load. The concrete strain at failure exceeded 0.003. The steel strain exceeded 0.01, confirming that the excess ultimate flexural strength over calculated values was due to strain hardening of the reinforcement.

Control of cracking

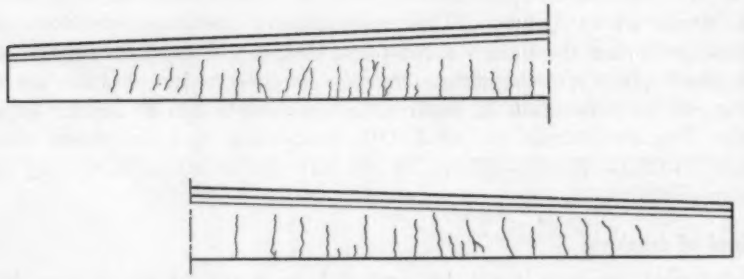
Reinforced concrete must be expected to crack at loads considerably below the full design service load. For a modular ratio $n = 30{,}000/f_c'$, tensile stress in the concrete at the tension steel level is $f_t = f_s f_c'/30{,}000$. Hence, if the modulus of rupture of the concrete is $f_t = 0.15\ f_c'$, cracking must be expected at a tensile steel stress $f_s = 4500$ psi. It is known that, for a given beam, the width of cracks is essentially proportional to steel stress, and it has been customary to limit crack width by limiting the allowable steel stress to 20,000 psi even when beam strength requirements would permit a higher steel stress.

The high strength reinforcement selected for the prototype girders has a minimum yield point of 70,000 psi. For a load factor of 1.8, the tension steel stress at service load is then 40,000 psi. Accordingly, it was necessary to develop other means of crack control than that of limiting the steel stress. The slender T-shaped cross section of the girders was chosen to reduce costs and dead weight and to realize a graceful architectural appearance. In addition, it was an important function of the slender section to obtain a crack pattern consisting of numerous very fine cracks rather than a few wide cracks.

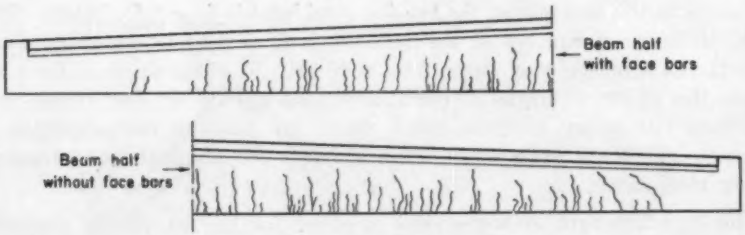
The width of cracks was measured with a graduated microscope in both girder tests, and the location and extension of cracks were noted at several load levels. The crack patterns at loads near that corresponding to the prototype service load (20.2 kips) are given in Fig. 7.

The width of flexural cracks in the constant moment region at midspan at a level 0.75 in. above the bottom of the girders is given in Fig. 7 as a function of steel stress. Both the maximum crack width and the average width are given. Cracks near the beam ends, including diagonal cracks, were always narrower than those near midspan. Comparison is made with the results of an auxiliary group of tests involving three 8 x 16-in. rectangular beams loaded at third-points of an 11-ft span.

It is seen in Fig. 7 that a change from rectangular to T-shaped cross section has a pronounced effect on both the maximum and the average crack width. At the prototype service load steel stress of 40,000 psi, the average crack width for the test girders is 0.001 to 0.002 in., which is the crack width for the rectangular beams at the customary steel stress of 20,000 psi. To control



Girder 1, P=20.7 Kips



Girder 2, P=23.2 Kips

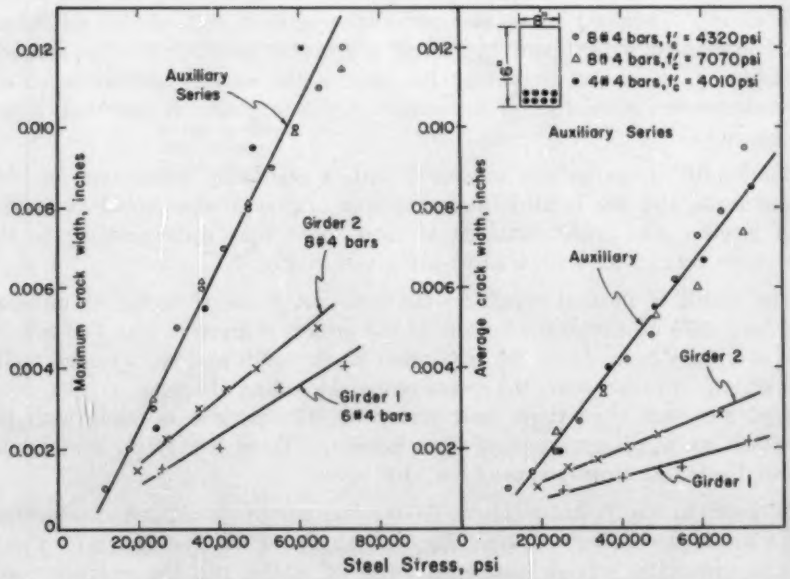


Fig. 7—Crack studies

cracking, therefore, the selection of a narrow girder width is an effective alternate in design to the customary measure of limiting steel stress.

To determine their effect on cracking, face bars were provided only in one half of Girder 2. Fig. 8 gives crack width cumulated for all cracks from

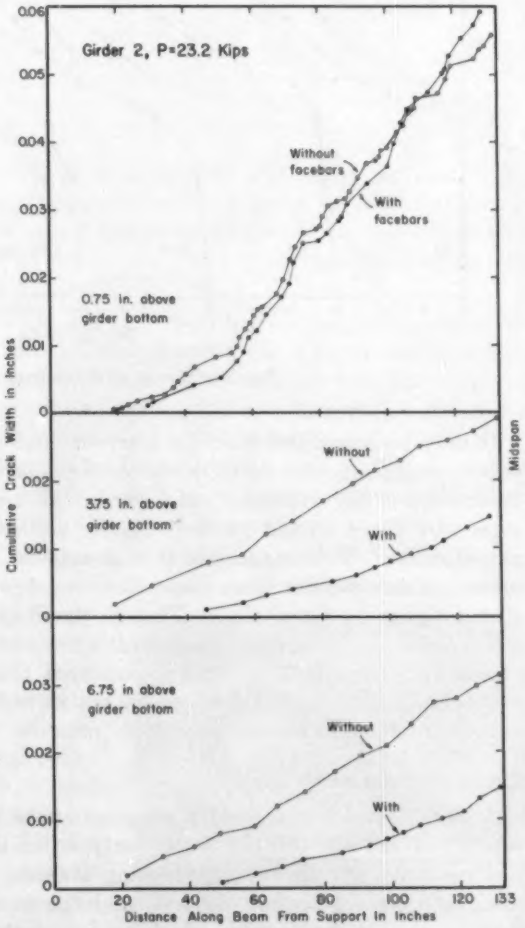


Fig. 8—Crack control by face bars

the girder end to midspan. At a level 0.75 in. above the bottom of the girder, the curve for the girder half with face bars practically coincides with that for the half without face bars. At the higher levels, however, Fig. 8 shows that the face bars reduced the cumulative crack width substantially. The face bars effectively control cracking in the beam web, and they were used in the entire span of the prototype girders.

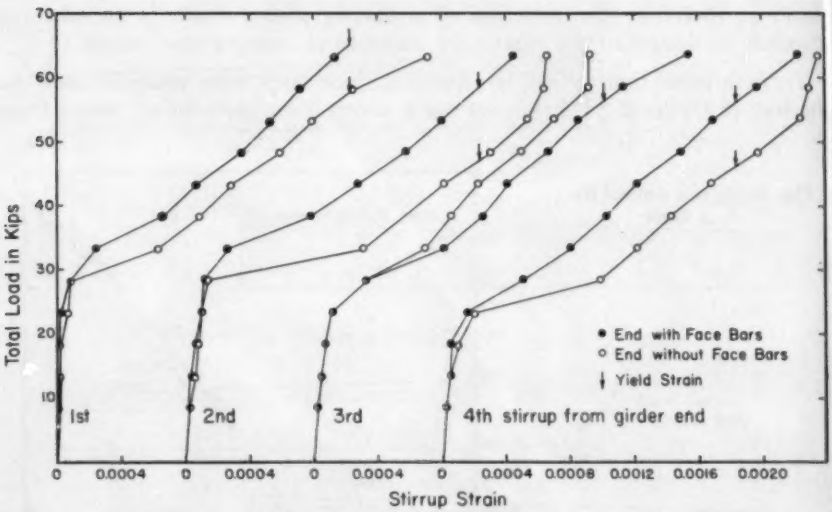


Fig. 9—Stirrup strain measurements

It may be concluded that, for a service-load steel stress of 40,000 psi, the prototype girders must be expected to be extensively cracked. By the slender T-section and the additional crack control afforded by face bars, these cracks at service loads should be individually so narrow that they will not be objectionable. To substantiate this, it may be noted that 76-ft roof girders somewhat less slender than those discussed herein were built in Stockholm after a design by Granholm.* Though the design steel stress at service load was 47,000 psi, measured crack width 6 years after construction did not exceed 0.004 in. It is generally considered that a crack width of 0.01 in. represents a safe limit below which cracks will not impair the beauty or durability of reinforced concrete structures.

Stirrup reinforcement

Girders 1 and 2 sustained a shearing stress at their ends, calculated by Eq. (2), of 723 and 980 psi, respectively, when the ultimate flexural strength was reached. At these high shearing stresses, there was no indication of shear or diagonal tension distress, and the inclined stirrups aided by face bars limited diagonal crack width to a few thousandths of an inch. For the prototype girders, therefore, a 300 psi design shearing stress at service load is entirely satisfactory, in spite of the fact that the 240 psi maximum value given by Section 803(c) is exceeded.

Electric strain gages were attached to four stirrups at each end of Girder 2 at middepth of the web. The gages were waterproofed and embedded.

*Granholm, H., "Modern Developments in Reinforced Concrete Design and the Influence of Research," *Proceedings*, Building Research Congress, London, 1951, Division 1, pp. 51-56.

TABLE 2—YIELD LOADS FOR END STIRRUPS

Stirrup No. from girder end	P_{yield}, kips		$P_{yield}/(36.6$ kips)	
	With face bars	Without face bars	With face bars	Without face bars
1st	65	58	1.78	1.58
2nd	58	47	1.58	1.28
3rd	56	61	1.53	1.67
4th	56	46	1.53	1.26
Average			1.60	1.45

The strain data are given in Fig. 9 as a function of total girder load. The stirrup strains were small until diagonal cracking began at a load of about 20 kips. The strains then increased, but more slowly for the beam half with face bars. The girder loads at which the yield strain was reached for the various stirrups are given in Table 2.

For the 4.25-in. stirrup spacing and the 55,000 psi stirrup yield point used, Eq. (3) gives an average V' of 18.3 kips between the support and first load point, and a total girder load of 36.6 kips. The ratio between the observed stirrup yield load and this calculated value of 36.6 kips averages about 1.5 in Table 2. This indicates that the stirrups carried 2/3 of the total shear when they yielded. The ultimate load for Girder 2 was 67.2 kips, and at this load the stirrups carried only 36.6/67.2 = 55 percent of the total shear. Even so, there was no indication of shear or diagonal tension distress.

To facilitate fabrication, intermediate grade stirrups with a minimum yield point of 40,000 psi are used in the prototype. These tests indicate that such stirrups may safely be designed by Section 804(d), but to carry 2/3 of the total service load shear throughout the girder span at a stirrup unit stress of 20,000 psi. The ultimate flexural strength of a prototype girder so designed can be fully developed without distress due to shear. It seems entirely satisfactory in this case to use inclined stirrups without welding them to the longitudinal steel, in spite of the fact that Section 802(a) calls for welding.

On the other hand, since no shear distress was observed in either of the two girder tests, it is entirely possible that stirrups designed to carry only the excess over a 90 psi shearing stress permitted on an unreinforced web

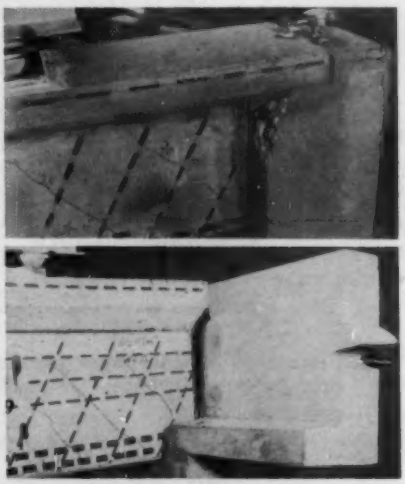


Fig. 10—Girder-column connection after test. Top—Girder 1; bottom—Girder 2

could suffice. It is the opinion of the writers, however, that any major roof girder, any beam with high strength reinforcement, and any beam with an unusually thin web should be provided with some web reinforcement throughout its span. This is particularly so for precast members because unforeseen erection loads, such as torsion, could seriously damage an unreinforced web.

Girder-column connections

Both girders were tested to failure in flexure with no indication of structural distress in the girder-column connection. Even the large rotations after the girders reached the plastic stage of large deflection were accommodated adequately. As shown in Fig. 10, bearing of the flange of Girder 1 caused local spalling of the column corner. This took place at service load, and could have been prevented by caulking rather than grouting the space between the flange and the column. For Girder 2 and for the prototype, however, the girder flange was discontinued at the face of the spandrel beams as seen in the lower photograph of Fig. 10. The performance of this latter connection was excellent in all respects.

CONCLUSIONS

Since the 1956 ACI Building Code reflects the state of reinforced concrete design as represented by the best practice of the day, it was considered that any departure from the Code provisions should be well substantiated.

On the basis of tests of two 0.38-scale model girders, it was found entirely safe and satisfactory in the present particular case to depart in design criteria from the Code provisions as follows:

(1) The 60,000 psi maximum value for stress in reinforcement at ultimate strength as given by Section A603(e) was increased to 70,000 psi.

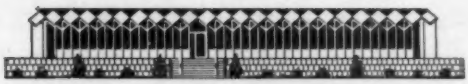
(2) The 240 psi maximum value given by Section 803(c) for unit shearing stress at service load of beams with stirrup reinforcement only was increased to 300 psi.

(3) The provision of Section 802(a) that inclined stirrups must be "welded or otherwise rigidly attached to the longitudinal steel" was waived.

(4) It was considered desirable in the present case to use a more conservative stirrup design procedure than that given by Sections 801 and 802. Two-thirds of the total shear was considered carried by stirrups throughout the girder span, even where the shearing stress "permitted on the concrete of an unreinforced web" was not exceeded.

(5) Face bars were provided to control cracking in the girder web.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.



structural research for better design

Title No. 55-32

shearing strength of PRESTRESSED LIFT SLABS*

By A. C. SCORDELIS,† T. Y. LIN,‡ and H. R. MAY§

A research investigation on the ultimate shearing strength of reinforced and prestressed concrete lift slabs included the testing of 15 slabs, 12 of which were prestressed with unbonded cables. All specimens were 6 ft square and had thicknesses of 6, 8, or 10 in. The slabs were supported along all four edges and centrally loaded.

Major variables were concrete strength, amount of prestressing or reinforcing steel, amount of initial prestress, size of steel collars, thickness of slab, and amount of collar recess.

A comparison of the test results with expressions for ultimate shearing strength of reinforced concrete slabs proposed by Elstner and Hognestad and by Whitney indicate that these expressions, with proper interpretation, may also be used for the prestressed slabs included within this series of tests.

IN RECENT YEARS the lift-slab method of construction has become a popular erection technique for buildings of the flat slab type. These slabs may be made of reinforced concrete or of prestressed concrete. The latter type has the advantage that deflection, which may be excessive in reinforced concrete slabs, can be minimized or even nullified by prestressing. The load on a lift slab is transferred to a column by means of a steel collar. This collar becomes an integral part of the slab at the time of casting and is fastened to the column, by welding or some other means, after the slab has been lifted to its final position.

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Present design of flat slabs with respect to shear is based on only a limited amount of factual knowledge. For reinforced concrete lift slabs most designers follow current specifications similar to those given in ACI 318-56 for shear design of flat slabs. For prestressed slabs, present practice varies considerably but generally consists of keeping the punching shearing stress at the

TABLE 1—DESCRIPTION OF TEST SPECIMENS

Group	Slab No.	Reinforced or Prestressed	Collar size c, in.	Total thickness t, in.	Collar recess, in.	Average d,* in.	Average p, percent	Effective prestress in steel f_s, ksi	Average prestress in concrete, psi	f'_c, psi
I	S-1	R	13	6	–	4.25	2.500	—	—	2813
	S-2	R	13	6	–	4.25	2.500	—	—	4059
II	S-4	P	13	6	1	3.63	0.452	165.0	450	3956
	S-5	P	13	6	–	3.00	0.547	91.8	250	2935
	S-6	P	13	6	–	3.63	0.452	91.8	250	4700
	S-7	P	13	6	–	4.38	0.562	125.0	500	2890
	S-8	P	13	6	–	3.63	0.452	183.6	500	4348
III	S-9	P	16	6	–	3.63	0.452	91.8	250	4376
	S-10	P	16	6	–	3.63	0.452	183.6	500	4668
IV	S-11	P	13	10	–	7.63	0.215	183.6	300	5125
	S-12	P	13	10	2	7.63	0.215	183.6	300	4919
	S-13	P	13	10	4	7.63	0.215	183.6	300	5228
V	S-14	P	13	8	–	5.63	0.293	183.6	375	4800
	S-15	P	13	8	2	5.63	0.293	183.6	375	5121
VI	S-16	R	18	8	–	6.50	1.517	—	—	3329

*Effective depth for flexure. To obtain effective depth for shear at collar subtract collar recess from these values.

TABLE 2—PROPERTIES OF CONCRETE MIXTURES

Mix. No.	Nominal strength, psi	Water-cement ratio		Mix ratio by weight SSD	Percent sand by weight	Cement factor, sacks per cu yd
		Gal. per sack	By weight			
1	3000	7.6	0.67	1:3.15:3.55	47.0	5.1
2	4500	5.5	0.49	1:2.17:3.20	40.5	6.3
3	4500	5.0	0.44	1:1.78:2.93	37.8	7.0

TABLE 3—AVERAGE VALUES OF CONCRETE PROPERTIES

Group	Slab No.	Mix No.	f'_c, psi	f'_t,* psi	E_c,† psi	Kelly ball penetration, in.	Age at test, days
I	S-1	1	2813	462	2.37×10^6	2.9	14
	S-2	2	4059	463	3.13×10^6	1.9	14
II	S-4	2	3956	571	3.06×10^6	2.1	14
	S-5	1	2935	455	2.75×10^6	2.0	14
	S-6	3	4700	456	3.52×10^6	2.3	13
	S-7	1	2890	488	2.66×10^6	1.6	13
	S-8	3	4348	605	3.37×10^6	2.3	14
III	S-9	3	4376	623	3.55×10^6	2.3	14
	S-10	3	4668	590	3.13×10^6	2.3	14
IV	S-11	3	5125	667	2.78×10^6	2.1	14
	S-12	3	4919	507	2.69×10^6	2.2	14
	S-13	3	5228	519	2.88	2.3	14
V	S-14	3	4800	561	3.46×10^6	2.2	14
	S-15	3	5121	472	3.45×10^6	2.2	14
VI	S-16	1	3329	517	2.89×10^6	1.5	14

*From 6 x 6 x 20-in. beams loaded at third points of 18-in. span.
†Secant modulus of elasticity at 1000 psi, from 6 x 12-in. cylinders.

edge of the collar below some arbitrary assigned value. This allowable value may range from 0.04 f_c' to 0.06 f_c'.

Little experimental work has been done to determine the shearing strength of such slabs. It was the purpose of this study to investigate the ultimate shearing strengths of prestressed concrete slabs (also some reinforced concrete slabs) with lift-slab collars and to compare experimental results for shearing strength with those indicated by various analytical expressions.

Scope

While present design practice bases the allowable shearing stress on only one variable, the cylinder strength, f_c', it is generally agreed that the ultimate shearing strength is also dependent on several other variables. The most important of these variables appears to be the distribution and relative magnitude of shearing and normal stresses on the critical section. Since in slabs the normal stresses are produced by flexural action, the shearing strength is therefore a function of the shear to moment ratio. In prestressed slabs the amount of prestressing will also have an effect on this variable. Another important factor to be considered is the amount of cracking that has taken place just prior to failure. This affects the net section available to resist shear at failure.

The foregoing effects as well as other practical design problems were studied by including the following variables in the experimental program: (a) concrete strength, (b) amount of prestressing or reinforcing steel, (c) amount of initial prestress, (d) size of steel collars, (e) thickness of slab, and (f) amount of collar recess.

Fifteen slabs were tested to ultimate failure. All specimens were 6 ft square and had thicknesses of 6, 8, or 10 in. Slabs were supported along all four edges and centrally loaded.

Fourteen slabs with lift-slab collars were tested. Of this group 12 were prestressed using unbonded cables and two were made of conventional reinforced concrete. In addition one reinforced concrete slab with a concrete column stub and shear reinforcement was tested.

EXPERIMENTAL PROGRAM

Description of test slabs

The test slabs were divided into six groups. A description of each specimen may be found in Table 1 and in Fig. 1, 2, 3, and 4. All slabs were 6 ft square. Outer dimensions of the square steel collars used were either 13 or 16 in. Concrete strengths varied from 2800 to 5200 psi. Average initial prestress for the slab concrete covered a range between 250 and 500 psi. Slab thicknesses were 6, 8, or 10 in. with collar recesses of 0, 1, 2, or 4 in.

Group I—The two slabs of this group, S-1 and S-2, were identical reinforced concrete slabs except for concrete strength. They were 6 in. thick with 13-in. steel collars. Fig. 1 shows a sketch of these slabs indicating the reinforcement used. Reinforcement was selected so as to give equal moment resistance in both directions. Bars were cut off at the edge of the column opening.

Group II—This group consisted of five prestressed concrete slabs, S-4 through S-8, which were all 6 in. thick with 13-in. collars. The variables consisted of concrete strength,

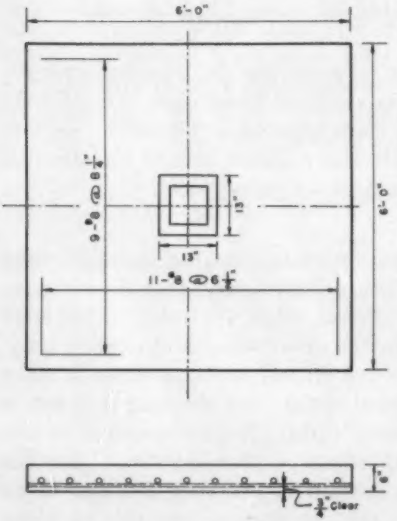

Fig. 1—Layout of reinforced concrete lift slabs S-1 and S-2

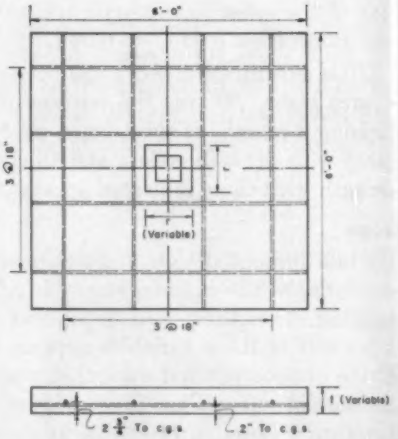

Fig. 2—Layout of prestressed concrete lift slabs (S-4, S-6, and S-8 through S-15)

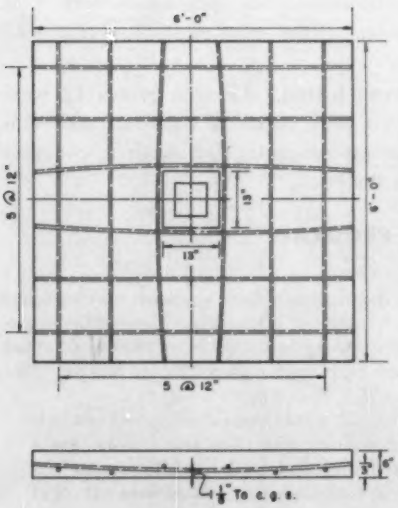

Fig. 3—Layout of prestressed concrete lift slab S-7

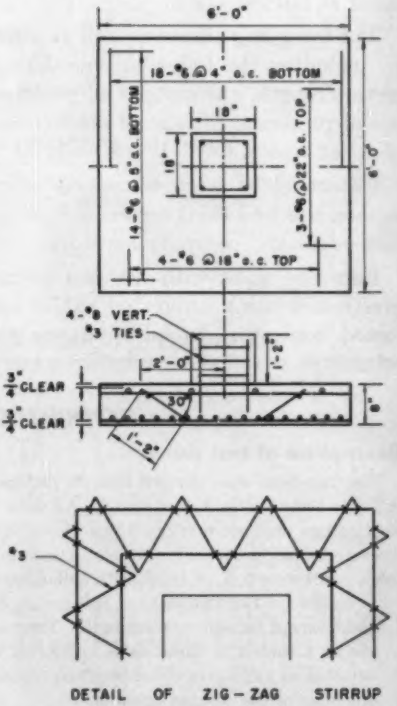

Fig. 4—Layout of special reinforced concrete slab S-16

amount of prestress, and arrangement of prestressing cables. The collars were all flush except that of S-4 which was recessed 1 in. Fig. 2 and 3 indicate the prestressing cable arrangements.

Group III—This group, consisting of S-9 and S-10, was similar to Group II with the exception that the collar size was 16 in. instead of 13 in. Arrangement of prestressing cables was similar to S-4, S-6, and S-8 and is shown in Fig. 2.

Group IV—Slabs S-11, S-12, and S-13 were all 10 in. thick and had 13-in. collars with recesses of 0, 2, and 4 in., respectively. Arrangement of prestressing cables was similar to Group III and is shown in Fig. 2.

Group V—Slabs S-14 and S-15 were 8 in. thick and had 13-in. collars with recesses of 0 and 2 in. respectively. Arrangement of prestressing cables was similar to Group III and is shown in Fig. 2.

Group VI—This group consisted of a single special slab, S-16, which was a full scale model simulating a reinforced concrete slab and column from a building designed by the Division of Architecture, State of California. Details of this slab are shown in Fig. 4. The slab was 8 in. thick. A "zig-zag" shear head constructed of #3 bars was placed concentrically around the column stub.

Materials

Forms were stripped 3 to 7 days after casting. The specimens were cured moist for 7 days using damp burlap and left air dry until testing at an age of 13 or 14 days. Prestressing was done the day prior to testing.

Santa Cruz Type I portland cement was used throughout the tests. The cement was purchased in one lot of 200 sacks and stored in steel drums until used. The fine aggregate used was Elliot S.E. 80 sand. The average fineness modulus was 3.20. The coarse aggregate was Elliot ¼ in. to ¾ in. gravel having an average fineness modulus of 6.59. Absorption for both

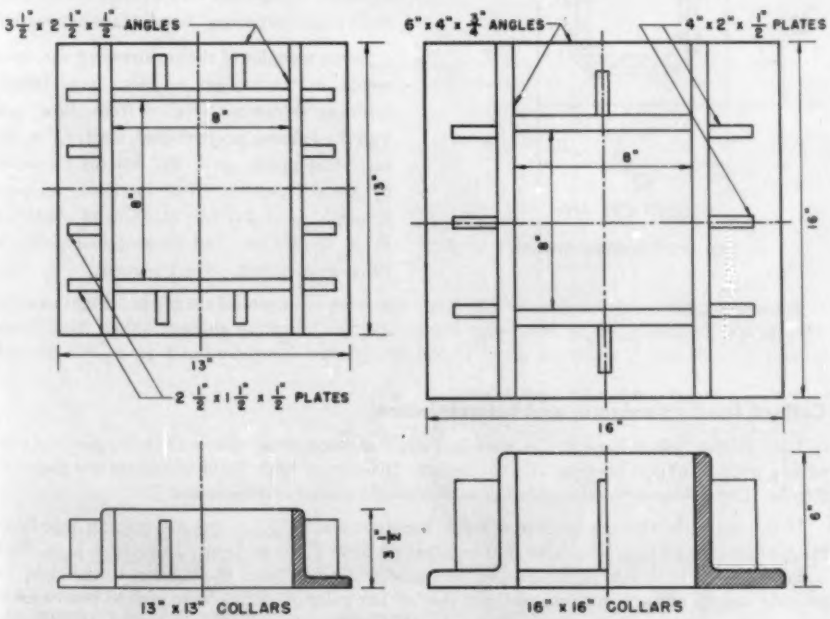

Fig. 5—Lift-slab collars

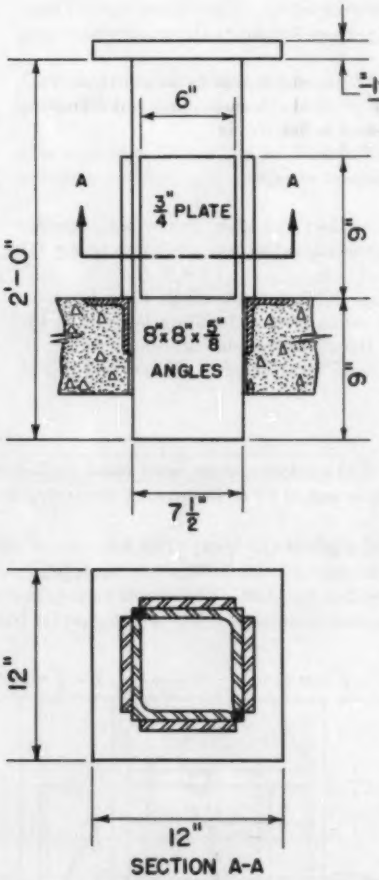

Fig. 6—Loading column

the fine and coarse aggregates was about 1.5 percent by weight. The specific gravities were 2.66 for sand and 2.69 for gravel.

Concrete mixtures were designed for 14-day strengths of 3000 and 4500 psi. Two different mixtures were used for the 4500-psi concrete. Water-cement ratios were 0.49 and 0.44 for the 4500-psi concrete mixtures and 0.67 for the 3000-psi concrete. Mix proportions are given in Table 2.

Control specimens consisted of four 6 x 12-in. cylinders and three 6 x 6 x 20-in. beams for each slab. They were cured in the same manner as the slabs. Cylinders and beams were tested on the same day as the corresponding test slab. Average values of compressive strength and modulus of elasticity obtained from the cylinders and modulus of rupture obtained from the control beams are presented in Table 3.

The prestressing cables each contained six ¼-in. diameter cold drawn high tensile steel wires. The wires were coated with a special asphaltic compound and wrapped with sisalkraft paper to prevent bonding to the concrete.

Seven samples of the prestressing wire were tested in tension on a 10-in. gage length. Average values determined from these tests were as follows: proportional limit, f_{pl} = 146 ksi; yield point, f_y = 217 ksi (as measured by the 0.2 percent offset method); ultimate strength, f_u = 247 ksi; modulus of elasticity, E_s = 26,300 ksi; and percent elongation in 10-in. gage length = 4.4 percent.

Reinforcing bars conformed to ASTM A 305 and were of intermediate grade. Eight samples were tested in tension on an 8-in. gage length and the following average values were determined: f_y = 50 ksi; f_u = 80 ksi; E_s = 28,500 ksi; percent elongation in 8 in. = 24.2 percent.

Collars, loading column, and support frame

The lift-slab collars used in the tests had outer dimensions of either 13 in. square or 16 in. square with a column opening of 8 in. square. Details of both types of collars are shown in Fig. 5. The collars were salvaged after each test and reused several times.

To simulate the column used in lift-slab construction, a 7½ in. square column was fabricated from two 8 x 8-in. angles welded together to form a box section. Lug plates ¾ in. thick were welded to each side of the column to transfer the load from the column to the slab. To provide nearly uniform bearing on each side of the column, shims were placed between the column lug plates and the slab collar. A spherical bearing block was placed between the column and the head of the testing machine. The loading column is shown in Fig. 6.

To approximate the conditions existing in continuous structures a simple support at the perimeter of the test slabs was selected. The position of this support approximates the line of zero moment around an interior column in a typical lift-slab structure. To provide this support a steel frame was constructed of 8 in. wide flange beams. The open frame was designed to provide a view of the bottom of the slab during testing. Wheels permitted the frame, with mounted slab, to be rolled into position under the testing machine after which the wheels could be removed. A 1 x 2-in. strip of oak wood was bolted to the top of the frame to allow some flexibility for the support. Hydrostone, a capping compound, was placed between the slab and support to ensure uniform bearing at the start of each test. No effort was made to hold down the slabs which, when loaded, tended to deflect upward at the corners.

Loading and instrumentation

Cable prestress was applied by means of a 30-ton capacity hydraulic jack which had been calibrated. Prestressing was always done 1 day prior to testing. The cables were overstressed 10 percent for a 2 to 3 min period, then released to a 5 percent overstress which was assumed to be the anchorage takeup loss. Creep and shrinkage for a 1-day period was considered negligible.

Load was applied with a 4,000,000-lb universal testing machine. The least reading of the machine in the range used was 500 lb. The capacity of the machine was much greater than needed, but it was necessary to use this machine because of the required clearance between screws necessary for the test assembly.

The slabs were loaded in 8 to 25 increments of load up to failure. Strains and deflections were measured after each loading increment.

Strains in the concrete were measured by 12 SR-4, type A-1 strain gages for each slab. Gage locations are shown in Fig. 7. Additional SR-4, type A-5 gages were placed on the reinforcing bars of slabs S-1 and S-2. The strain data proved to give useful information in only a few cases and are therefore not reported in this paper.

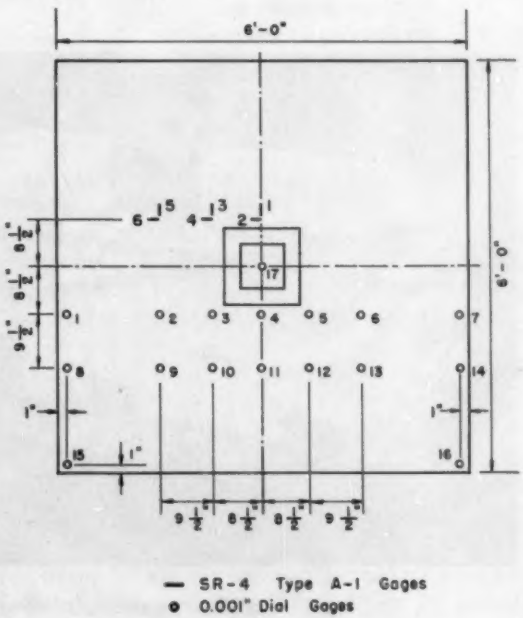

Fig. 7—Location of SR-4 gages and deflection gages

SR-4 gages No. 7-12 are located on the bottom of the slab directly underneath No. 1-6, respectively

Deflections were measured along two lines with fourteen 0.001-in. dial gages attached to a bridge to give the slab profile after each loading increment. The 0.001-in. dial gages also measured the deflections of the column and two slab corners. Fig. 7 shows the locations of the dial gages. The corners were free to lift during all tests.

The photographs of Fig. 8 show the test assembly and the instrumentation.

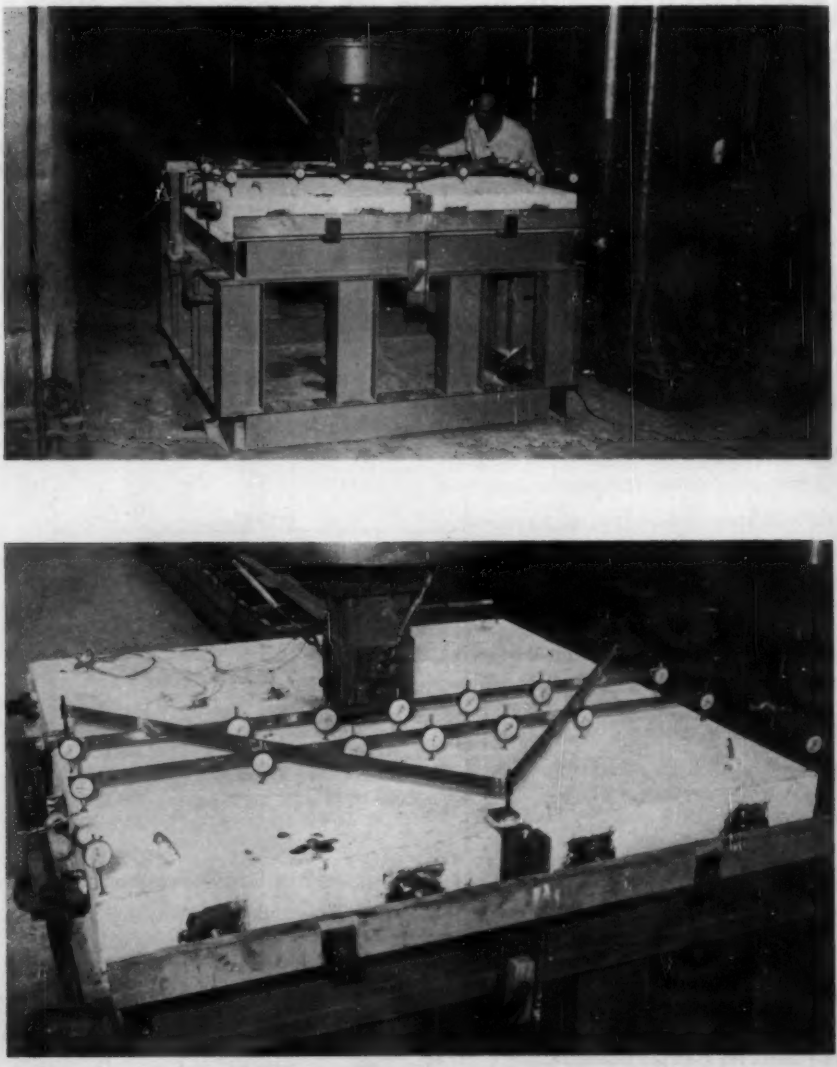

Fig. 8—(top) Test assembly; (bottom) Instrumentation

NOTATION

The letter symbols used in this paper are generally defined when they are introduced. The most frequently used symbols are listed below for convenient reference:

a = width of slab
b = perimeter of vertical shear area at critical section
d = average effective depth of tensile reinforcement, distance from compression surface of a slab to the plane common to each layer of tensile reinforcement
E_c = secant modulus of elasticity at 1000 psi obtained from 6 x 12-in. concrete cylinders
E_s = modulus of elasticity of steel
f_c' = compressive strength of 6 x 12-in. concrete cylinders
f_e = effective unit stress in psi in prestressing steel prior to loading
F_e = effective total force in lb in prestressing steel prior to loading of slab
f_{pl} = proportional limit of prestressing steel
f_t' = modulus of rupture of concrete
f_u = ultimate strength of prestressing or reinforcing steel
f_y = yield point of prestressing or reinforcing steel
j = ratio of internal moment arm to effective depth, assumed equal to $\frac{7}{8}$
m_u = ultimate resisting moment per unit width
P = load
P_{shear} = calculated ultimate shearing load
P_{flex} = calculated ultimate flexural load
P_{test} = measured ultimate load
p = average percentage of tensile reinforcement
q = tensile reinforcement index

$$= \frac{pf_y}{f_c'}$$

r = side dimension of square loaded area
s = cable spacing in in.
t = total thickness of slab
V = shearing force
V_1 = shearing force corresponding to v_1
V_2 = shearing force corresponding to v_2
V_3 = shearing force corresponding to v_3
v = shearing stress
v_1 = shearing stress computed at a distance d from the edge of the loaded area
v_2 = shearing stress computed at the edge of the loaded area
v_3 = shearing stress computed at a distance $d/2$ from the edge of the loaded area
ϕ_s = ratio of P_{shear}/P_{flex}

ANALYTICAL STUDIES

At present there is no design specification available for computing the allowable shear load on a prestressed concrete flat slab. For reinforced concrete flat slabs, however, the 1956 ACI Building Code[1] prescribes that the shearing unit stress existing in a flat slab be computed by the formula:

$$v_1 = \frac{V}{bjd} \qquad (1)$$

in which V is the total shear acting on vertical sections which follow a periphery, b, at a distance, d, beyond the edges of the column or column capital and parallel or concentric with it; d is specified as the effective depth of the tensile reinforcement and j is normally taken as $\frac{7}{8}$. The ACI Code further stipulates that the stress as computed by Eq. (1) shall not exceed an allowable shearing stress which is given as a function of the cylinder stress, f_c', and the

amount of bending reinforcement passing over the column or column capital. In applying Eq. (1) to lift slabs, designers normally take the outer edge of the steel collar as synonymous with the edge of the column or column capital in the ACI definition.

Studies made by Elstner and Hognestad[2,3] indicated that the above procedure yielded a highly variable factor of safety between allowable and ultimate strengths when applied to slab tests conducted by them and to footing tests

TABLE 4—CALCULATED VALUES FOR ULTIMATE FLEXURAL STRENGTH
See Eq. (4), (5), (6), (7)

Group	Slab No.	f_s, ksi	f_p,* ksi	f'_c, psi	p, percent	q	d,† in.	m_u, kip-in. per in.	P_{flex}, kips
I	S-1	—	48.0	2813	2.500	0.426	4.25	16.2	136.3
	S-2	—	48.0	4059	2.500	0.295	4.25	17.8	149.3
II	S-4	165.0	223.0	3956	0.452	0.255	3.63	11.3	83.5
	S-5	91.8	193.7	2935	0.547	0.361	3.00	7.51	63.0
	S-6	91.8	193.7	4700	0.452	0.186	3.63	10.1	84.9
	S-7	125.0	207.0	2890	0.562	0.403	4.38	17.0	142.5
	S-8	183.6	230.4	4348	0.452	0.240	3.63	11.8	98.7
III	S-9	91.8	193.7	4376	0.452	0.200	3.63	10.2	90.5
	S-10	183.6	230.4	4668	0.452	0.223	3.63	11.9	106.1
IV	S-11	183.6	230.4	5125	0.215	0.097	7.63	27.2	227.8
	S-12	183.6	230.4	4919	0.215	0.101	7.63	27.1	202.5
	S-13	183.6	230.4	5228	0.215	0.095	7.63	27.2	180.0
V	S-14	183.6	230.4	4800	0.293	0.141	5.63	19.6	164.3
	S-15	183.6	230.4	5121	0.293	0.132	5.63	19.7	140.2
VI	S-16	—	48.0	3329	1.517	0.219	6.50	11.3	250.0

*$f_p = 157 + 0.40 f_c$ for prestressed slabs.
†Effective depth for flexure. To obtain effective depth for shear at collar subtract collar recess from these values.

TABLE 5—TEST RESULTS

Ultimate shearing stress $v = \dfrac{P_{test}}{\frac{7}{8} bd}$

v_1—at a distance d from the edge of the collar
v_2—at zero distance from the edge of the collar
v_3—at a distance $d/2$ from the edge of the collar

Group	Slab No.	P_{test}, kips	d (at collar), in.	r, in.	v_1 psi	v_2 psi	v_3 psi	f'_c, psi	v_1/f'_c	v_2/f'_c	v_3/f'_c
I	S-1	105.0	4.25	13	328	543	407	2813	0.116	0.194	0.145
	S-2	109.0	4.25	13	341	563	423	4059	0.084	0.139	0.104
II	S-4	80.0	2.63	13	476	666	558	3956	0.121	0.169	0.141
	S-5	60.0	3.00	13	300	440	357	2935	0.104	0.150	0.122
	S-6	78.5	3.63	13	304	475	372	4700	0.065	0.101	0.079
	S-7	121.5	4.38	13	366	610	455	2890	0.127	0.211	0.157
	S-8	99.5	3.63	13	386	602	472	4348	0.089	0.139	0.109
III	S-9	105.0	3.63	16	356	517	421	4376	0.081	0.118	0.096
	S-10	118.0	3.63	16	400	581	474	4668	0.086	0.125	0.101
IV	S-11	225.0	7.63	13	298	648	408	5125	0.058	0.127	0.080
	S-12	171.5	5.63	13	358	670	466	4919	0.073	0.136	0.095
	S-13	109.0	3.63	13	422	660	516	5228	0.081	0.126	0.099
V	S-14	168.0	5.63	13	351	655	457	4800	0.073	0.138	0.095
	S-15	120.0	3.63	13	465	725	569	5121	0.091	0.142	0.111
VI	S-16	250.0*	6.50	18	273	468	343	3329	0.082	0.141	0.103

*250.0 kips was total test load. In computing ultimate shearing stresses in concrete use 191.6 kips which is based on yielding shear reinforcement taking 58.4 kips.

reported by Richart.[4] Recently Whitney[6] has made an extensive study of the same test data. These studies resulted in proposed new methods for determining the allowable and ultimate shearing strength of reinforced concrete slabs.

The ultimate shearing strength of a reinforced or a prestressed concrete slab is primarily a function of the net area available to resist shear at the critical section and the ultimate shearing stress available at that section. Several complications arise, however, in trying to predict the magnitude of these quantities.

(*a*) *Where should the critical section be taken?* (*b*) *What proportion of the total uncracked depth should be used in computing the net area available to resist shear?* (*c*) *What are the variables affecting the ultimate shearing stress and how can the stress distributions be predicted in a particular case?*

The location of the critical section recommended by the ACI Code is at a distance d from the edge of the loaded area; Elstner and Hognestad recommend that it be taken at the edge of the loaded area; Whitney proposes that it be taken at a distance $d/2$ from the edge of the loaded area. The actual distance to be taken cannot yet be determined analytically and must be determined empirically by correlation with test results.

The proportion of the total uncracked depth to be used in computing the net area available to resist shear is also a difficult quantity to ascertain since it is dependent on the amount and location of flexural cracking that has taken place just prior to ultimate failure. Certainly for slabs with high shear-moment ratios the amount of flexural cracking will be smaller than for slabs with low shear-moment ratios. From a practical standpoint a fixed depth equal to the effective depth at the critical section might be used in computing a nominal stress and the ultimate shearing stress could then be determined empirically by including the shear-moment ratio as a variable.

Analytical study of test slabs

From the preceding discussion it is evident that at the present time a method for predicting the ultimate shearing strength of reinforced and prestressed concrete can only be developed by applying empirical approaches to the results obtained from tests.

For comparative purposes the ultimate shearing stress for the test slabs was calculated at a distance d from the edge of the collar, v_1; at the edge of the collar, v_2; and at a distance $d/2$ from the edge of the collar, v_3. These values are shown in Table 5 under test results. In all cases v was taken as equal to $P_{test}/\frac{7}{8}\,bd$. A study of these results indicates that v or v/f_c' cannot be considered a constant at any of the three sections.

The empirical approaches that were applied to the test data are discussed below.

1956 ACI Code method

The allowable shearing stress for flat slabs as specified by the 1956 ACI Code is $0.025\,f_c'$, but is not to be greater than 85 psi. Values of allowable

TABLE 6—COMPARISON OF ANALYTICAL METHODS FOR CALCULATING SHEARING STRENGTH OF TEST SLABS

Group	Slab No.	1956 ACI method			Elstner-Hognestad method [Eq. (3)]					Whitney method [Eq. (8) and (10)]							Method using Eq. (11)			
		Allow v_i, psi	P_{allow} kips	$\frac{P_{test}}{P_{allow}}$	P_{flex}, kips	$\frac{P_{test}}{P_{flex}}$	$\frac{d_o = P_{shear}}{P_{flex}}$	P_{shear}, kips	$\frac{P_{test}}{P_{shear}}$	m_u, kip-in. per in.	d at collar, in.	l_s, in.	$\frac{m_u}{d^2}\sqrt{\frac{d}{l_s}}$, psi	v_u, psi	P_{shear}, kips	$\frac{P_{test}}{P_{shear}}$	F_s, kips	s, in.	P_{shear}, kips	$\frac{P_{test}}{P_{shear}}$
I	S-1	70.3	22.6	4.64	136.3	0.770	0.725	98.9	1.061	16.2	4.25	29.5	343	357	105.0	1.000				
	S-2	85.0	27.2	4.00	149.3	0.739	0.753	112.4	0.971	17.8	4.25	29.5	375	381	112.0	0.974				
II	S-4	85.0	14.2	5.60	83.5	0.959	0.800	66.7	1.200	11.3	2.63	29.5	490	468	76.9	1.041	48.6	18	72.0	1.111
	S-5	73.3	14.6	4.10	63.0	0.953	1.011	63.7	0.943	7.5	3.00	29.5	266	299	57.5	1.042	27.0	18	61.4	0.977
	S-6	85.0	21.9	3.58	84.9	0.925	1.048	89.0	0.884	10.1	3.63	29.5	269	302	73.5	1.068	27.0	18	80.9	0.970
	S-7	72.2	24.1†	5.08	142.5	0.853	0.722	102.9†	1.181	17.0	4.38	29.5	342	356	108.5†	1.119	36.0	12	108.5	1.120
	S-8	85.0	21.9	4.55	98.7	1.009	0.919	90.8	1.096	11.8	3.63	29.5	314	336	81.0	1.229	54.0	18	106.4	0.935
III	S-9	85.0	25.1	4.19	90.5	1.160	1.142	103.3	1.017	10.2	3.63	28.0	279	309	88.0	1.191	27.0	18	100.6	1.044
	S-10	85.0	25.1	4.71	106.1	1.112	1.031	109.6	1.078	11.9	3.63	28.0	325	344	98.0	1.205	54.0	18	132.1	0.893
IV	S-11	85.0	64.0	3.49	227.8	0.990	0.905	205.9	1.094	27.2	7.63	29.5	237	278	175.0	1.288	54.0	18	225.6	0.997
	S-12	85.0	40.5	4.21	202.5	0.846	0.785	158.7	1.080	27.1	5.63	29.5	374	380	159.6	1.075	54.0	18	166.9	1.028
	S-13	85.0	21.9	4.96	180.0	0.606	0.646	116.2	0.938	27.2	3.63	29.5	726	645	155.0	0.703	54.0	18	106.9	1.020
V	S-14	85.0	40.5	4.12	164.3	1.022	0.899	147.9	1.137	19.6	5.63	29.5	270	302	127.0	1.322	54.0	18	166.8	1.010
	S-15	85.0	21.9	5.45	140.2	0.855	0.761	106.8	1.075	19.7	3.63	29.5	525	494	119.0	1.008	54.0	18	107.2	1.119
VI	S-16	83.3	58.5	3.27*	250.0	1.000	0.847	210.0‡	1.191	11.3	6.50	27.0	298	324	206.0	0.921*				

*See footnote in Table 5.
†Shear taken by parabolic cables neglected.
‡Shear taken by shear reinforcement included. (See Reference 2 for method of computation.)

shearing stress v_1, $P_{allow} = 7/8\ v_1\ bd$, and P_{test}/P_{allow} are given in Table 6. The ratio P_{test}/P_{allow} represents the factor of safety for the slabs tested.

As might be expected the application of this method to prestressed as well as reinforced concrete slabs yields a wide range of factors of safety, varying from about 3.3 to 5.6. Even for slabs S-1 and S-2, which were identical reinforced concrete slabs except for concrete strength, the factors of safety were 4.64 and 4.00 respectively.

Elstner-Hognestad method

In 1953 Hognestad,[3] recognizing that the ultimate shearing stress in footings and slabs was related to the flexural capacity of the specimen, reported on a re-evaluation and re-analysis of Richart's[4] footing tests. Based on this study Hognestad found the ultimate shearing strength of a variety of slabs could be expressed by the empirical equation:

$$v_2 = \frac{V}{7/8\ bd} = \left(0.035 + \frac{0.07}{\phi_o}\right) f_c' + 130 \qquad (2)$$

or rewriting:

$$\frac{v_2}{f_c'} = 0.035 + \frac{130}{f_c'} + \frac{0.07}{\phi_o} \qquad (2a)$$

in which v_2 represents the ultimate shearing stress computed at the perimeter of the loaded area, V is the total shearing force, b is the perimeter of the loaded area, and f_c' is the ultimate compressive strength of the concrete; ϕ_o is defined as the ratio of P_{shear} to P_{flex} where P_{flex} is the ultimate flexural capacity the slab would have if it had not failed in shear.

In 1956 Elstner and Hognestad[2] reported on a continuation of the above study in which they gave the results of new tests performed on thirty-nine 6 ft square reinforced concrete slabs. These slabs were in general supported only at the edges and centrally loaded through a reinforced concrete column stub. To give better correlation for higher concrete strengths, Elstner and Hognestad revised Eq. (2a) for slabs without shear reinforcement to give:

$$\frac{v_2}{f_c'} = \frac{P_{shear}}{7/8\ bdf_c'} = \frac{333}{f_c'} + \frac{0.046}{\phi_o} \qquad (3)$$

Additional equations were presented for slabs with shear reinforcement. In computing the ultimate flexural strength of slabs, the yield line theory[5] was used. Application of Eq. (3) to Richart's column footing tests and the slab tests of Elstner and Hognestad gave values of P_{test}/P_{calc} ranging from 0.84 to 1.17 with average values near 1.0.

The application of Eq. (3) to the reinforced and prestressed concrete lift slabs tested in the present investigation gave good correlation with test results.

The results are shown in Table 6 which indicates a range of values of P_{test}/P_{shear} from 0.88 to 1.20 with an average value of 1.06 and a standard deviation of 0.096 for the 15 slabs. P_{shear} represents the theoretical value obtained for the test specimen.

In using Eq. (3) the computation of P_{flex}, the ultimate flexural capacity of the slab, presents some difficulties when applied to slabs with lift-slab collars and also when considering prestressed concrete slabs with unbound cables.

In the application of the yield line theory for the ultimate flexural strength of the test slabs the question arises of how much, if any, moment can be developed at the perimeter of the collar. In addition what is the effect on the magnitude of this perimeter moment due to recessing of the collar? Observation during the tests and a study of the test results seem to indicate that for flush collars the full moment can develop at the perimeter of the collar and for slabs with collars recessed an amount equal to about $0.5d$, where d is the effective depth for the slab with unrecessed collar, this perimeter moment diminishes to nearly zero. A study of collar strains measured by SR-4 gages attached to a collar flush with the concrete indicates that the collar transfers a large compressive force across the column opening. (Visual inspection of salvaged collars showed cracks in the weld at the joint of the vertical angle legs. These joints were rewelded with a heavier weld.) With large collar recesses the compression force transmitted was small. From these observations it was concluded that collars flush with the concrete developed the full moment capacity of the section at the perimeter of the collar and for collars recessed 0.5 d or greater no moment could be developed at the perimeter of the collar.

For the case of a simply supported square slab loaded through a square column with the corners free to lift, the ultimate flexural strength given by the yield line theory is:

$$P_{flex} = 8m_u\left[\left(\frac{1}{1-\frac{r}{a}}\right) - (3 - 2\sqrt{2})\right] \quad \ldots\ldots\ldots\ldots\ldots (4)$$

where m_u is the ultimate moment capacity per unit width of the slab and r/a is the ratio of the side dimension of the loaded area to the slab width. For lift slabs, r becomes the collar width, 13 or 16 in. in this investigation. Eq. (5) assumes that the moment, m_u, is developed along the diagonals and at the perimeter of the loaded area.

If no moment is developed at the perimeter of the loaded area, the ultimate flexural strength for a square slab with the corners free to lift becomes:

$$P_{flex} = 16\, m_u\,(\sqrt{2} - 1) = 6.627\, m_u \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

To give approximate values of P_{flex} for recesses of less than 0.5 d a straight line interpolation between the values given by Eq. (4) and (5) can be assumed,

e.g., for a recess of 0.25 d, P_{flex} would be 6.627 m_u plus one-half of the difference between Eq. (5) and (4) for the particular slab dimensions.

When applying the yield line theory to prestressed concrete slabs it must first be assumed that the prestressing steel has sufficient ductility for the full yield line pattern to form and second, when unbonded cables are used, a method for estimating the steel stress for the needed computation of m_u, the ultimate or yield moment, must be devised. The yield line theory can only be considered an approximate method when applied to the computation of the ultimate flexural strength of an unbonded prestressed concrete slab.

The value of the stress in the prestressing steel developed at ultimate moment was designated f_y. The values of P_{flex} calculated for the slabs tested, by using various expressions for the value of f_y which have been proposed for beams, were greatly below the test loads. This indicated that these expressions for beams, probably developed for much longer cables, were not applicable to the short cables used in the test slabs. A study of the test results, along with approximate computations for the cable elongation due to fiber strain, cracking, and deflection of the slab, showed that f_y for the test slabs could be approximated by the expression:

$$f_y = 157{,}000 + 0.4 f_e \qquad (6)$$

where f_e is the effective prestressing steel stress in psi prior to loading. It must be noted that Eq. (6) is purely empirical and is applicable only to the short cables in this test series. For longer cables as would be found in prestressed slab construction other relationships should be used which would depend on the situation considered. Once f_y has been established the yield moment per unit width may be computed by means of the following formula:

$$m_u = f_c' d^2 q\,(1 - 0.59q) \qquad (7)$$

in which $q = pf_y/f_c'$ and average values for p and d are used for the two layers of steel.

Using the method described above, values of P_{flex} for the tests slabs were calculated and these are tabulated in Table 4. Having obtained P_{flex}, values for P_{shear} were calculated by Eq. (3). Results are given in Table 6. It should be pointed out in using Eq. (3) that the value of P_{shear} obtained is relatively insensitive to changes in P_{flex}. A change in the value of P_{flex} used of 10 or 15 percent will only change the value of P_{shear} by 2 or 3 percent.

To show the effect of concrete strength, f_c', and ultimate shear-flexure ratio, ϕ_o, the values given in Table 6 were adjusted first to a common value of $f_c' = 3500$ psi and then to a common value of $\phi_o = 1.0$ using the equations

$$\frac{v_2}{f_c'}\ (\text{corrected to } f_c' = 3500) = \frac{v_2}{f_c'_{obs}} - \frac{333}{f_c'} + \frac{333}{3500}$$

$$\frac{v_2}{f_c'}\ (\text{corrected to } \phi_o = 1.0) = \frac{v_2}{f_c'_{obs}} - \frac{0.046}{\phi_o} + \frac{0.046}{1.0}$$

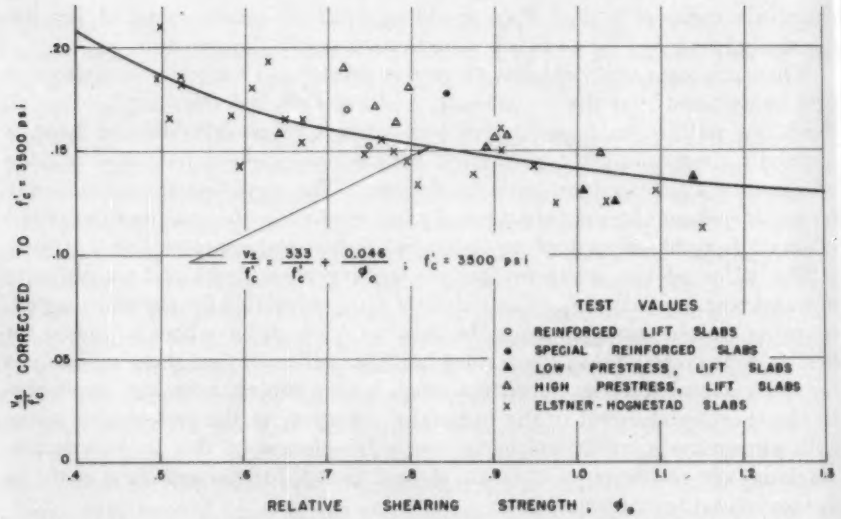

Fig. 9—Influence of shear-flexural strength ratio using Eq. (3)

These results are shown graphically in Fig. 9 and 10 which indicate a decrease in ultimate shearing strength, v_2/f_c' with increasing values of ϕ_o or f_c'.

The results obtained for the slabs tested by Elstner and Hognestad are also indicated on these figures for purpose of comparison. It can be noted that there is considerable scatter of the test data for these reinforced slabs as well as for the prestressed slabs.

Whitney method

In 1957 Whitney[6] proposed that the ultimate shearing strength of reinforced concrete flat slabs should be computed by the equation:

$$v_u = 100 + 0.75 \frac{m_u}{d^2} \sqrt{\frac{d}{l_s}} \quad \ldots\ldots (8)$$

in which v_u is the ultimate shearing stress at a distance $d/2$ from the face of the column, l_s is the distance from the face of the column to the load position, and m_u is the ultimate resisting moment per in. width of slab within the base of the pyramid of rupture. The m_u may be computed by Eq. (7) for under-reinforced slabs and by Eq. (9) when over-reinforced.

$$m_u = \frac{d^2 f_c'}{3} \quad \ldots\ldots (9)$$

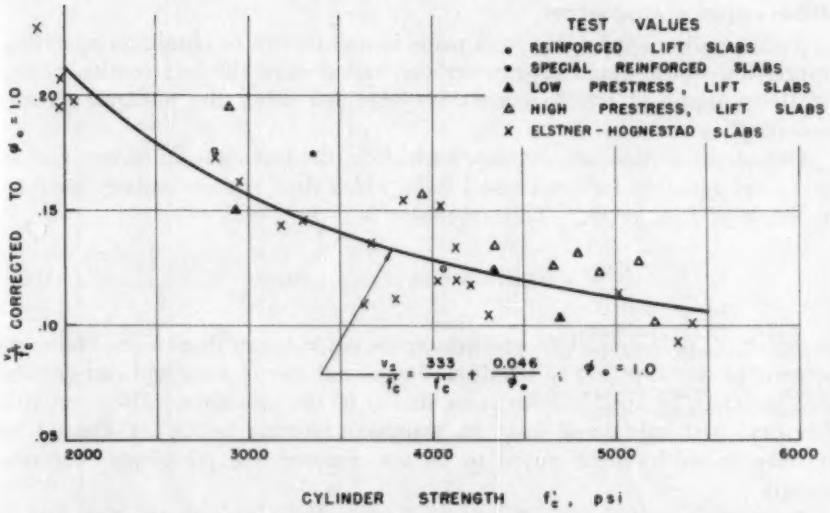

Fig. 10—Influence of cylinder strength using Eq. (3)

Having obtained the ultimate shearing stress from Eq. (8), the ultimate shearing strength of a flat slab, as governed by shear, is calculated by:

$$P_{shear} = v_u\, bd = 4\, v_u\, d\,(r + d) \qquad (10)$$

Whitney's proposed method was based on his analysis of the test data of Richart,[4] Elstner,[2] and Hognestad.[2,3]

The method described above was applied to the test data obtained in this investigation to ascertain if it would give satisfactory results when applied to prestressed as well as reinforced slabs. The results are shown in Table 6 which indicates a range of values for the slabs (excluding S-13 which had a 4-in. recess) from 0.92 to 1.32 for P_{test}/P_{shear} with an average value of 1.10 and a standard deviation of 0.123 for 14 slabs.

For slabs with recessed collars m_u was computed using the full effective depth neglecting the recess, while the d used in computing $(m_u/d^2)\sqrt{d/l_e}$ was taken as the effective depth at the collar. This procedure gave the best correlation for the test data.

The results obtained using Whitney's method are shown graphically in Fig. 11. The results of the Elstner-Hognestad test slabs are also shown. It can be seen that the results for the prestressed slabs generally fall within the limits of the data obtained for reinforced slabs. Modified forms of Eq. (8) could be proposed which would fit this test data equally well, but at best any of these can only be considered as estimates of the shear strength which may be useful for design purposes.

Other empirical approaches

Various other approaches were made in an attempt to obtain an analytical expression which would give better correlation with the test results. None of these improved on the correlation obtained using the methods already described.

One of the expressions developed which fit the test data quite well was an empirical equation for prestressed slabs which does not necessitate the computation of P_{flex} or m_u. This expression is as follows:

$$\frac{P_{shear}}{b\,d\,f_c'} = 0.175 - 0.0000242\,f_c' + 0.000020\,\frac{F_e}{s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which F_e is the effective prestress force per cable in lb and s is the cable spacing in in. Eq. (11) is based on a small number of tests and can only be considered to be applicable in cases similar to the test slabs. However, it is felt that with additional tests an expression similar to Eq. (11) could be developed which might prove to be the simplest for prestressed concrete design.

A comparison of the results obtained using Eq. (11) with the test data is given in Table 6. Values of P_{test}/P_{shear} range from 0.89 to 1.12 with an average value of 1.02 and a standard deviation of 0.075 for the 12 prestressed slabs. These results are indicated graphically in Fig. 12.

EXPERIMENTAL RESULTS

General behavior

All slabs were loaded to ultimate failure, which in all cases occurred by a final punching of the steel collar through the concrete. The manner of failure was sudden and violent in some cases and often accompanied by the dropping out from the bottom of the slab of a large amount of concrete.

In the case of the prestressed concrete slabs, the sequence of events during the test was generally as follows:

a. The first visible flexural cracks appeared at 40 to 60 percent of the ultimate load for the slabs with flush collars and at succeedingly higher percentages as the amount of collar recess was increased.
b. The corners of the slab lifted and this was generally followed by the appearance of vertical cracks at the edges of the slab a short distance in from the corners.
c. The flexural cracks on the bottom of the slab spread into a crack pattern and in most cases reached the corners of the slab; the amount of flexural cracking visible just prior to failure varied considerably.
d. The collar punched through with concrete dropping from the bottom of slab.
e. The failure cone was very flat and extended in some cases beyond the edge of the slab.
f. Upon release of the load the tension in the prestressing cables tended to put the bottom of the slab back into compression and in cases where a large amount of concrete had fallen from the bottom, leaving no compression area available, the slab buckled upwards.

The behavior of the three reinforced concrete slabs tested differed from the prestressed concrete slabs in that the first visible flexural cracks occurred

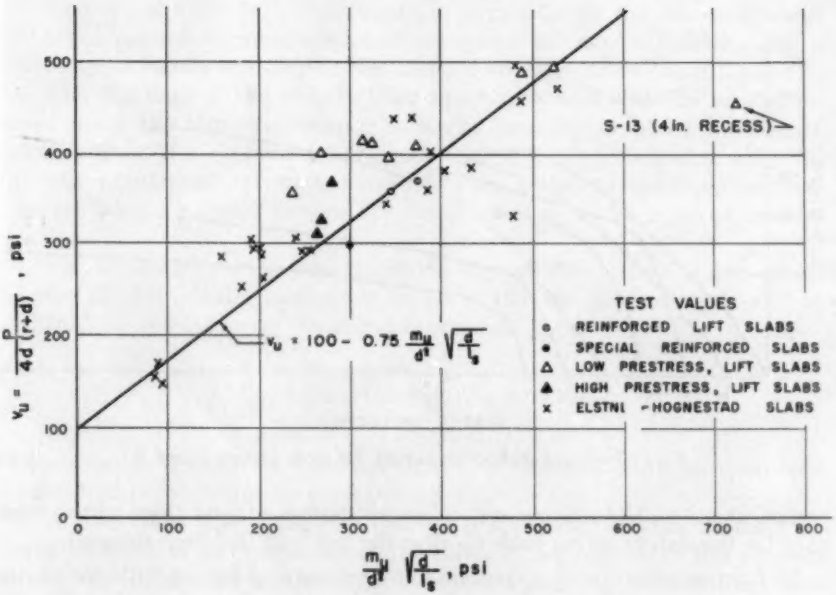

Fig. 11—Test values compared with Eq. (8)

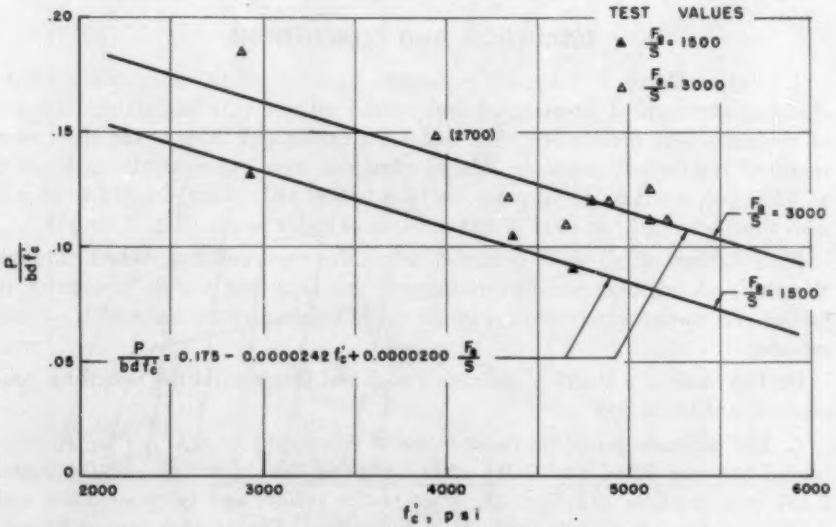

Fig. 12—Test values compared with Eq. (11)

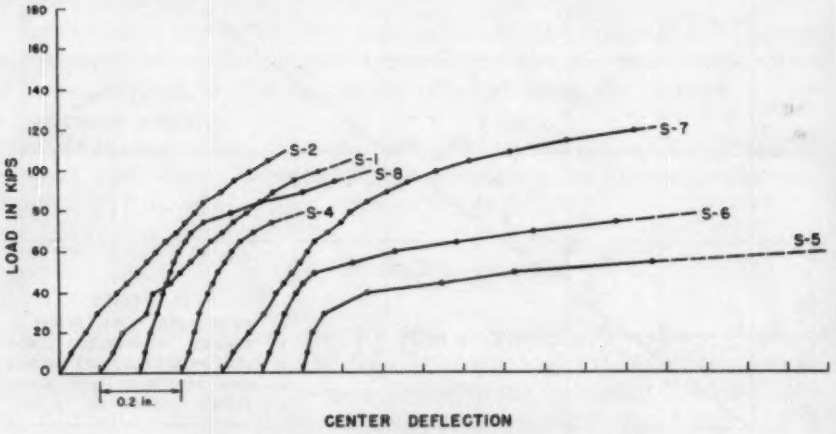

Fig. 13—Load-deflection curves for slab groups I and II

earlier, at about 15 to 25 percent of the ultimate load, and there was no tendency for the slab to spring back up after the test load had been released.

All failures were due to a combination of shear and flexure with one or the other dominating. By studying the load-deflection curves given in Fig. 13 and 14 one can estimate the relative amount of flexural cracking that took place prior to the failure in each case.

A detailed description of the behavior of each slab during testing may be found in Reference 7.

DISCUSSION AND CONCLUSIONS

A total of fifteen 6 ft square specimens were tested to study the ultimate shearing strength of prestressed and reinforced concrete lift slabs. Twelve of the slabs were prestressed with unbonded cables and three of the slabs were made of reinforced concrete. Major variables were concrete strength (2800 to 5200 psi), average initial prestress (250 to 500 psi), collar size (13 to 16 in.), slab thickness (6, 8, or 10 in.), and amount of collar recess (0, 1, 2, or 4 in.).

Final failure of all slabs occurred when the steel collar punched through the slab. A variable amount of flexural cracking was visible just prior to failure and was generally smaller in the thicker slabs and the slabs with column recesses.

On the basis of a study of the analytical and test results the following conclusions are advanced.

1. The ultimate punching shear stress as computed by $v/f_c' = P_{test}/7/8\ bd\ f_c'$ varied between 0.101 and 0.211 at the edge of the collar; between 0.079 and 0.157 at a distance $d/2$ from the edge of the collar; and between 0.058 and 0.127 at a distance d from the edge of the collar. Therefore it cannot be considered to be a constant at any one of these locations.

2. The use of 1956 ACI Code methods of calculating the allowable shear load when applied to prestressed and reinforced concrete lift slabs yields a wide range of factors of safety on the ultimate strength, from about 3.3 to 5.6.

3. The test data of this investigation agree quite well with the results obtained using the ultimate shearing strength formulas proposed by Elstner and Hognestad [Eq. (3)] and by Whitney [Eq. (8)]. These formulas will yield sufficient accuracy for prestressed concrete slabs as well as reinforced concrete slabs provided a suitable method is used in calculating the ultimate flexural capacity.

4. The ultimate shearing strength at the edge of the collar of a prestressed concrete lift slab similar to those included in this test program may also be predicted by the following empirical expression:

$$\frac{P_{shear}}{bd\,f_c'} = 0.175 - 0.0000242\,f_c' + 0.000020\,\frac{F_s}{s}$$

This expression is based on a limited amount of test data and additional tests are required to determine its general applicability.

5. A steel collar cast flush with the slab will develop the full ultimate resisting moment at the perimeter of the collar. The amount of this moment developed will decrease as the collar is recessed and will be essentially zero

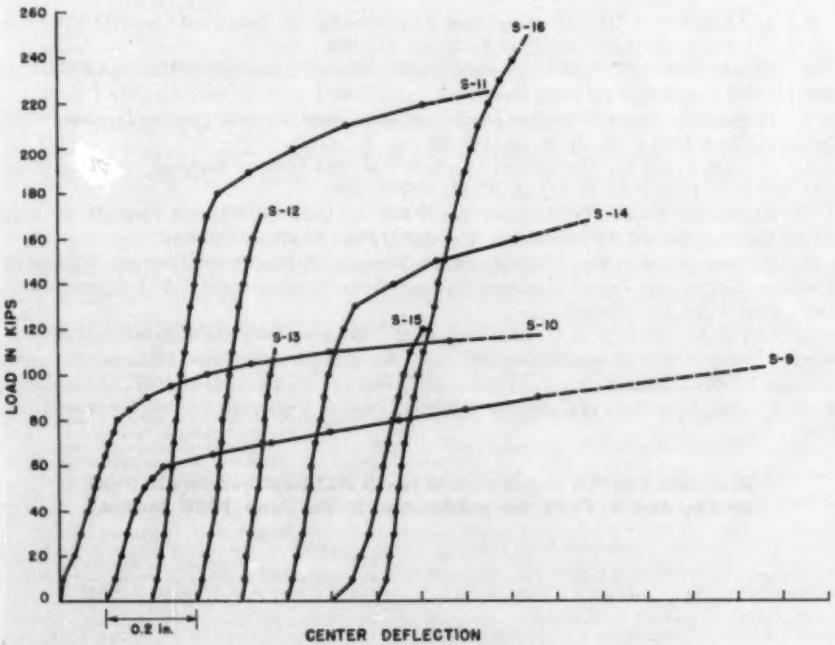

Fig. 14—Load-deflection curves for slabs in groups III-VI

when the collar has been recessed $0.5d$, where d is the effective depth for the slab with unrecessed collar.

6. Adequate provisions should be made in design so that ultimate flexural capacity will govern failure rather than ultimate shear capacity since a shear failure may be sudden and without warning.

ACKNOWLEDGMENTS

The program reported herein was conducted as a research project in the Structural Engineering Laboratory, Division of Civil Engineering, University of California. The project was sponsored by the Division of Architecture, Department of Public Works, State of California. The program was carried out during 1956 and 1957.

The Division of Architecture is under the direction of Anson Boyd, state architect. Special appreciation for their help and suggestions is due to C. M. Herd, chief construction engineer; Charles Peterson and M. W. Sahlberg, principal structural engineers; and A. H. Brownfield, supervising structural engineer, all of the Division of Architecture. S. A. Ravid and S. Y. Chang, graduate research assistants, University of California, participated in the conduct of the tests and the reduction and interpretation of the data.

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materials research . . . . .
air void series concluded

Title No. 55-33

Origin, Evolution, and Effects of the Air Void System in Concrete

*Part 4—The Air Void System in Job Concrete**

By RICHARD C. MIELENZ,† VLADIMIR E. WOLKODOFF,‡
JAMES E. BACKSTROM,§ and RICHARD W. BURROWS**

The air void system observed in concrete from engineering structures is comparable to that observed in concrete specimens prepared in the laboratory. The void system in non-air-entrained concrete varies widely, the observed specific surface ranging from 107 to 1111 in.$^{-1}$ In air-entrained concrete, the observed specific surface ranges from 615 to 1600 in.$^{-1}$ and the spacing factor ranges from 0.0023 to 0.0081 in. The results of the examination indicate that a satisfactory air void system in job concrete is assured if the recommendations of ACI 613-54 are followed.

Methods for microscopical measurement of the air void system in hardened concrete are described in an appendix to Part 4.

This paper concludes a series of reports on studies conducted in the Engineering Laboratories of the Bureau of Reclamation to describe and explain the development of the air void system in concrete. The first three parts of this series†† summarized the findings of laboratory tests and the conclusions developed from theoretical concepts. Part 4 presents data obtained by microscopical measurement of the parameters of the air void system in hardened concrete from structures and control cylinders from various projects. Included with Part 4 is an appendix describing microscopical methods for measuring the air void system in hardened concrete, with particular reference to the procedures employed in the study reported in this series of papers.

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††Mielenz, Richard C., Wolkodoff, Vladimir E., Backstrom, James E., and Flack, Harry L., "Origin, Evolution, and Effects of the Air Void System in Concrete. Part 1—Entrained Air in Unhardened Concrete," ACI JOURNAL, July 1958, *Proc.* V. 55, pp. 95-122.

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AIR VOID SYSTEM IN NON-AIR-ENTRAINED CONCRETE FROM STRUCTURES

Sections of drilled core and irregular pieces of non-air-entrained concrete from 20 structures were available in the laboratories. Of the structures represented 13 were dams, including several of major size, such as Hoover, Grand Coulee, Shasta, and Parker. Results of the microscopical analysis are presented in Table 1.

TABLE 1—VOID SYSTEM IN NON-AIR-ENTRAINED CONCRETE FROM STRUCTURES

Source		Microscopically determined void parameters				
Structure	Part of structure represented	Air content, percent	Average chord intercept, in.	Specific surface, α, in.$^{-1}$	Number of voids intercepted per in.	Spacing factor, $\bar{L}$, in.
La Prele Dam, Wyoming	Mass concrete	0.8	0.0049	816	1.64	0.0121
Stewart Mountain Dam, Arizona	Mass concrete	1.4	0.0315	127	0.45	0.0514
Shasta Dam, California	Mass concrete	0.3	0.0036	1111	0.86	0.0113
Black Canyon Dam, Idaho	Mass concrete	0.7	0.0091	440	0.78	0.0200
Grand Coulee Dam, Washington	Mass concrete	0.6	0.0127	315	0.46	0.0298
Bonneville Dam, Oregon-Washington	Mass concrete	2.9	0.0062	645	4.78	0.0091
American Falls Dam, Idaho	Mass concrete	1.5	0.0041	976	3.62	0.0087
Wu Sheh Dam, Formosa (China)	Mass concrete	3.1	0.0263	152	1.19	0.0357
Grassy Lake Dam, Wyoming	Mass concrete	1.3	0.0143	280	0.94	0.0284
Rosa Dam, Washington	Mass concrete	1.1	0.0090	444	1.22	0.0195
Parker Dam, California-Arizona	Mass concrete	0.3	0.0080	500	1.25	0.0251
Buck Dam, Virginia	Mass concrete	0.2	0.0050	800	0.40	0.0210
All American Canal, California	Canal lining	1.6	0.0073	548	2.19	0.0139
Tornillo Canal, Texas	Canal lining	5.7	0.0100	400	5.67	0.0106
Hoover Dam, Arizona-Nevada	Spillway	0.9	0.0083	482	1.09	0.0193
	Underbed below terrazzo floor of monument	3.8	0.0162	247	2.33	0.0229
Flume and wasteway, Owyhee Project, Oregon	Lining	1.0	0.0087	460	1.10	0.0211
Building, Ent Air Force Base, Colorado Springs, Colo.	Roof cornice	3.3	0.0090	444	3.74	0.0137
Street pavement, Kimball, Neb.	Pavement	3.7	0.0217	184	1.70	0.0299
Parking apron, Casper, Wyo.	Pavement A	2.2	0.0375	107	0.59	0.0614
	B	1.9	0.0267	150	0.72	0.0468
Residence, Wheatridge, Colo.	Patio slab	3.7	0.0341	117	1.09	0.0469
Average		2.0	0.0137	443	1.94	0.0247

The parameters are highly variable from structure to structure. The range of the several parameters is as follows:

Parameter	Range		
	Minimum	Maximum	Average
Air content, percent by volume	0.2	5.7	1.9
Specific surface α, in.$^{-1}$	107	1111	443
Spacing factor $\bar{L}$, in.	0.0087	0.0614	0.025

The air content of non-air-entrained mass concrete in the dams in the United States for which data are available is indicated to range from 0.2 to 2.9 percent, the usual range being 0.3 to 1.5 percent. For other types of non-air-entrained concrete, the air content ranges from 1.0 to 5.7 percent.

TABLE 2—VOID SYSTEM IN AIR-ENTRAINED CONCRETE FROM STRUCTURES

Source			Microscopically determined void parameters				
Structure	Part of structure represented		Air content, percent	Average chord intercept, in.	Specific surface, α, in.$^{-1}$	Number of voids intercepted per in.	Spacing factor, $\bar{L}$, in.
Hungry Horse Dam, Montana	Mass concrete	Exterior	3.6	0.0044	909	8.18	0.0052
		Interior	5.6	0.0047	851	11.91	0.0032
Canyon Ferry Dam, Montana	Mass concrete	Exterior	3.3	0.0046	870	7.18	0.0054
		Interior	3.5	0.0053	755	6.61	0.0056
Davis Dam, Nevada-Arizona	Mass concrete	Exterior	2.9	0.0049	816	5.92	0.0057
		Interior	4.8	0.0057	702	8.42	0.0045
Angostura Dam, South Dakota	Mass concrete		3.1	0.0036	1111	8.61	0.0042
Friant-Kern Canal, California	Mass concrete		3.7	0.0036	1111	10.33	0.0039
Poudre Supply Canal, Colorado	Canal lining		4.2	0.0065	615	6.34	0.0081
Tecolote Tunnel, California	Canal lining		10.6	0.0036	1111	29.05	0.0023
Carlsbad Caverns Project, New Mexico	Tunnel lining		5.0	0.0037	1081	13.23	0.0043
South St. Vrain Project, Colorado	Highway curb and gutter*		4.9	0.0044	909	11.07	0.0050
Big Bend National Park Project, Texas	Highway bridge footing*		5.3	0.0025	1600	21.22	0.0028
Hugo Wild Horse Project, Colorado	Highway culvert*		2.1	0.0032	1250	6.83	0.0056
Evans-LaSalle Project, Colorado	Highway pavement†		4.3	0.0043	930	9.99	0.0053
Denver-Broomfield Project, Colorado	Highway pavement†		4.7	0.0044	909	10.71	0.0052
Paonia North Project, Colorado	Highway pavement†		4.5	0.0033	1212	13.63	0.0041
Limon-Genoa Project, Colorado	Highway pavement†		4.3	0.0031	1290	13.87	0.0038
Breed Underpass, Breed, Colorado	Highway pavement†		3.9	0.0035	1143	11.14	0.0045
Building, Buffalo, N. Y.	Highway underpass†		2.4	0.0036	1111	6.66	0.0058
Railroad bridge‡	Structural member		5.6	0.0059	678	9.44	0.0065
	Prestressed deck slab		3.5	0.0055	727	6.22	0.0075
Average			4.4	0.0043	986	10.75	0.0049

*Control cylinders compacted by hand rodding, supplied by the Bureau of Public Roads, U. S. Department of Commerce.
†Control cylinders compacted by hand rodding, supplied by the Colorado Department of Highways.
‡Slab cast and prestressed at a plant in Denver, Colo.

The range established for the specific surface came as somewhat of a surprise. For several structures, α* is in the range above 800 in.$^{-1}$ Nevertheless, in all instances, $\bar{L}$ is greater than is desirable for satisfactory resistance to freezing and thawing.

AIR VOID SYSTEM IN AIR-ENTRAINED CONCRETE FROM STRUCTURES

Sections of drilled core and miscellaneous pieces of air-entrained concrete were available from ten structures, and control cylinders from several highway projects were kindly supplied by the Bureau of Public Roads and the Colorado Department of Highways. Results of the microscopical analysis are summarized in Table 2.

For mass concrete from several major dams, the air content ranges from 2.9 to 5.6 percent; α ranges from 702 to 1111 in.$^{-1}$; and $\bar{L}$ ranges from 0.0032 to 0.0057 in. For the other types of concrete represented by the samples,

*As in previous parts of the series, A = volume of voids, expressed as a fraction of the volume of the concrete; α = specific surface of voids; and $\bar{L}$ = computed spacing factor.

the air content ranged from 2.1 to 10.6 percent; α ranged from 615 to 1600 in.$^{-1}$; and $\bar{L}$ ranged from 0.0023 to 0.0081 in.

The investigation demonstrates that the parameters of the air void system in air-entrained concrete in structures fall within the range characteristic of laboratory prepared air-entrained concrete.

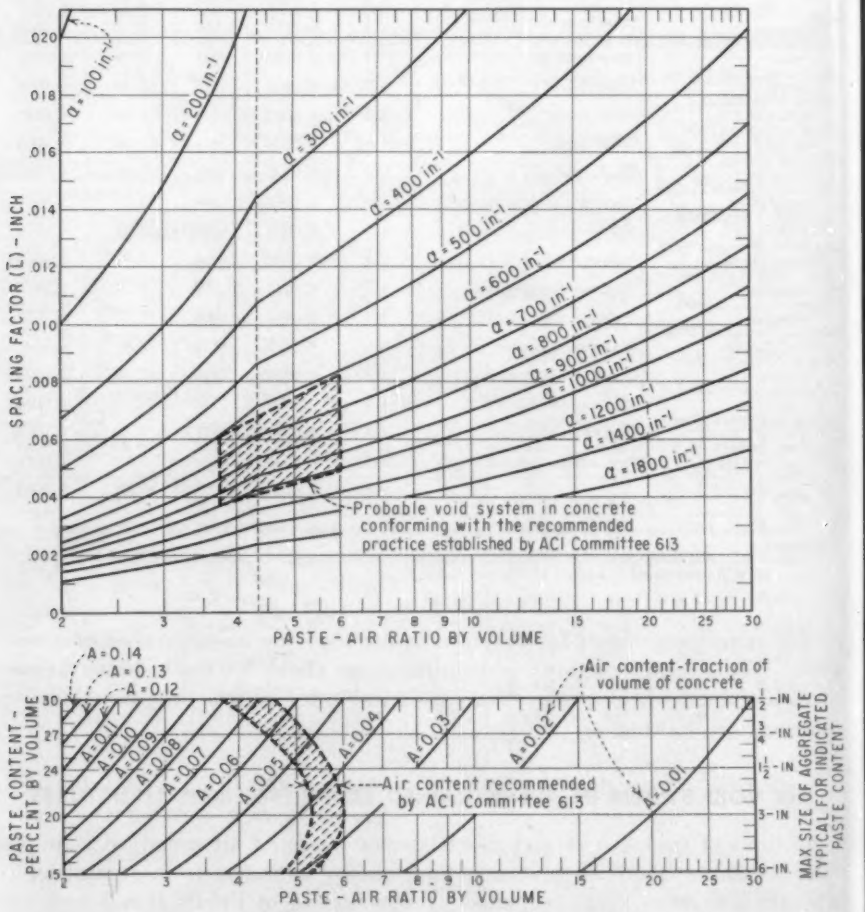
SPACING FACTOR ($\bar{L}$) - INCH
PASTE - AIR RATIO BY VOLUME
α = 100 in⁻¹
α = 200 in⁻¹
α = 300 in⁻¹
α = 400 in⁻¹
α = 500 in⁻¹
α = 600 in⁻¹
α = 700 in⁻¹
α = 800 in⁻¹
α = 900 in⁻¹
α = 1000 in⁻¹
α = 1200 in⁻¹
α = 1400 in⁻¹
α = 1800 in⁻¹
Probable void system in concrete conforming with the recommended practice established by ACI Committee 613
A = 0.14
A = 0.13
A = 0.12
A = 0.11
A = 0.10
A = 0.09
A = 0.08
A = 0.07
A = 0.06
A = 0.05
A = 0.04
A = 0.03
A = 0.02
A = 0.01
Air content-fraction of volume of concrete
Air content recommended by ACI Committee 613
PASTE CONTENT - PERCENT BY VOLUME
MAX SIZE OF AGGREGATE TYPICAL FOR INDICATED PASTE CONTENT
PASTE - AIR RATIO BY VOLUME

Fig. 1—Probable void system in air-entrained concrete conforming with ACI 613-54

To determine the spacing factor for a void system in a concrete from microscopically determined parameters, select the calculated or estimated paste content on the ordinate of the lower diagram and follow horizontally to the proper air content or paste-air ratio of the concrete. Now move vertically to the upper diagram, which correlates the specific surface of the voids α and the spacing factor $\bar{L}$. If α has been measured or is estimated, the value of $\bar{L}$ is indicated at the ordinate. Conversely, α is indicated on the curves at any selected values of $\bar{L}$ and paste-air ratio.

It is instructive to examine "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)"[2]* in the light of these data. In Fig. 1, the recommended average total air content for air-entrained concrete is superimposed upon a graphical solution of Powers' spacing factor formulae.[1] As may be seen in the lower diagram, this range of air content yields values of paste-air ratio from about 3.75 to about 6.00 for concrete containing aggregate whose maximum size is ½ in. to 6 in. Correlating this range with the empirically established, usual range of α, the usual range of spacing factor $\overline{L}$ is indicated in the upper diagram to be 0.004 to 0.008 in., a value consistent with that found by microscopical analysis of the specimens of job concrete for which air entrainment was specified.

In other words, the requirements of ACI 613-54 are such that they usually assure development of a void system adequate to protect the cement paste against failure in natural freezing and thawing.

Air content of the finished concrete in excess of that recommended will assure lower (and hence more desirable) values of $\overline{L}$, but possibly with unnecessary loss of strength of the concrete. In other words, referring to the upper part of Fig. 1, a locus representing a series of similar concretes containing progressively greater proportions of an air-entraining admixture would extend to the left and steeply downward from the starting point.

Air content in the finished concrete less than that recommended may or may not correlate with higher values of $\overline{L}$. If the low air content arises because of use of an inadequate proportion of air-entraining admixture or if the air-entraining admixture is inferior, $\overline{L}$ will be excessive. However, if a satisfactory air-entraining agent is used but the air content is reduced inordinately by the process of compaction, $\overline{L}$ will be adequate because the low value of A will be compensated by the high value of α. A data point representing this concrete would move to the right and only slightly downward as vibration or tamping is increased in intensity and duration.

CONCLUSIONS

1. The parameters of the air void system in air-entrained concrete in structures fall within the range characteristic of laboratory prepared air-entrained concrete.

2. A satisfactory entrained air void system in job concrete is assured if "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" is followed, provided the air-entraining admixture meets the requirements of ASTM C 260, "Specifications for Air-Entraining Admixtures for Concrete."† Under these conditions, the specific surface of the air void system may be expected to range from 600 to 1100 in.$^{-1}$ and the computed spacing factor from about 0.004 to 0.008 in.

*References are numbered consecutively beginning with Part 1 of this four-part series.
†*ASTM Book of Standards, Part 3*, 1955, p. 1279.

3. Additional investigations of the void system in air-entrained concrete are needed to extend the data presented herein, especially to establish the variation in the parameters of the void system caused by normal variation of materials and conditions on the job, such as variation in proportions of aggregates, cement, water, and admixtures; source and grading of aggregates; source and shipment of cement, particularly air-entraining cements; ambient temperature; and workmanship, especially placing and finishing operations.

APPENDIX—OPTICAL MEASUREMENT OF AIR VOIDS IN HARDENED CONCRETE

By Richard C. Mielenz and V. E. Wolkodoff

METHODS OF MEASUREMENT

Optical measurement of the air void content and the dimensions and spacing of air voids in hardened concrete is accomplished by one of three general procedures, namely: (1) point count, (2) areal traverse, and (3) linear traverse. The data included in this series of reports on the void system in hardened concrete were obtained primarily by the linear traverse method. Several determinations were made by the areal traverse method. The point count method was not used in this study.

Point count method

The point count method was developed for mineralogic analysis of rock[28,29] but can be used to determine the air void content of concrete.[30,31,32] The method is as accurate as other microscopical methods of analysis, and the air content can be determined more rapidly than by the areal or linear traverse methods. The procedure involves examination of a finely ground surface or thin section under a microscope at regularly spaced points. For determination of air void content of concrete, $A = S_v/S_T$ where A = volume of air expressed as a fraction of the volume of the concrete; S_v = number of points falling within the section of an air void; and S_T = total number of points observed.

One writer has used the point count method to considerable extent in evaluating the air void system of concrete. Both A and n (number of voids intercepted per inch of traverse) are determined experimentally. The device used is a specially constructed, manually-operated mechanical stage with total lateral movement of 5 in., a count being made at each rotation of the lead screw. A mechanical stop is engaged at completion of each turn. Points are spaced at 0.05 in. Lines of traverse usually are spaced 0.20 in. apart, although they can be set at any desired interval. For most concrete 1000 counts are sufficient to yield an indicated air content within 0.5 percentile of a value obtained at by counting 2000 points. Typical results are as follows:

Specimen	Indicated air content, percent by volume, for given number of counts					
	100	500	1000	1500	2000	2500
1	1.0	1.8	1.0	0.8	0.8	0.8
2	1.0	3.4	2.6	2.2	1.9	
3	0.0	1.4	1.4	1.3	1.3	
4	3.0	3.0	4.3	4.1	4.6	
5	4.0	5.0	4.0	4.0	4.7	
6	5.0	3.8	4.3	3.9	3.8	
7	5.4	5.7	4.8	4.1	4.5	
8	5.2	4.0	3.5	3.3	3.7	

The prime factors controlling the validity of the result are the perfection of the finished surface on the concrete specimen and the area of the concrete over which the traverse is made. The variations indicated in the above tabulation for each specimen are primarily the result of variation of the void system in the concrete.

The point count method ordinarily does not yield data on the size or spacing of the voids, but by a simple supplement to the point count, the specific surface of the voids α and Powers' spacing factor $\bar{L}$[1] can be determined. If the number of voids intercepted by the line of traverse is recorded during the point count analysis, the number of voids per inch of traverse n can be computed. Since

$$A = n\bar{l}$$

and

$$\bar{l} = \frac{A}{n}$$

where $\bar{l}$ is the average chord intercept of the voids intersected by the line of traverse, α may be computed by Willis' formula[8]

$$\alpha = \frac{4}{\bar{l}}$$

Also, if the paste content of the concrete is known, the spacing factor $\bar{L}$ can be computed from Powers' formulae.[1] The results so determined are the same as those established by the linear traverse method, within the limits of experimental error of the methods. Determination of n should be based upon a traverse of at least 100 in. for concrete containing 1½-in. maximum size aggregate. A shorter traverse will suffice for a uniform concrete containing aggregate of smaller maximum size.

Areal traverse method

The areal traverse method involves the measurement of the area occupied by sections of air voids on plane surfaces of concrete. The method was employed first by Verbeck,[32] who measured the area of the void sections in camera lucida drawings of the finely ground surfaces. Photographs also may be used for this purpose.[34] Lord and Willis[16] and Warren[35] accomplished areal traverses by counting and measuring the diameter of void sections within a known area on a plane ground surface of concrete, either by direct observation or by measurement on a photograph. The areal traverses reported in Parts 2 and 3 of the series of papers were made by direct microscopical measurement of void sections on finely ground surfaces, at least 1500 voids being counted for each specimen.

Mathematical treatment of the data resulting from this procedure is developed by Lord and Willis.[16] In this analysis,

$$A = \frac{\text{area occupied by void sections}}{\text{area of the surface traversed}}$$

Linear traverse method

The linear traverse method is based upon procedures described by Rosiwal in 1898. In the Rosiwal analysis, the length of traverse across all mineral grains of each species intercepted along a line or series of lines on a plane surface, such as a thin section, is measured and the proportion by volume of each mineral in the rock is calculated by dividing the total traverse distance across all the grains of the mineral by the length of the traverse as a whole. In a linear traverse analysis of hardened concrete to determine air content and the parameters of the void system, only the void sections are distinguished from the remainder of the concrete. For this analysis, $A = n\bar{l}$, where n = number of voids traversed per unit of traverse and $\bar{l}$ = average void intercept or chord length on the line of traverse.

Lord and Willis[18] have demonstrated that the void size distribution can be obtained from a modification of the Rosiwal analysis. In this procedure, the chord lengths are tabulated in narrow intervals of size, and from these data the probable distribution of true diameters and an average void diameter are calculated. Moreover, Philleo[26] has proved mathematically that the diameter of a void having the mean surface area of the void system or the diameter of a void having the mean volume of the voids in the system can be calculated even if only the fraction of the chord intercepts falling in the smallest size group is known, in addition to the data obtained routinely from the linear traverse.

The linear traverse method developed for concrete by Brown and Pierson[12] and treated mathematically by Powers,[1] Willis,[8] and Lord and Willis[18] is the most generally applicable procedure for evaluating the air void system in concrete. The Bureau of Reclamation equipment is essentially a duplicate of the apparatus developed by the Missouri State Highway Department from information supplied by the Portland Cement Association (for an illustration see Reference 37, Fig. A, page 124-2). The Missouri State Highway Department supplied working drawings; modifications were made in the drive mechanism and clutch design during development of our apparatus.

The apparatus comprises a lathe driven by a 25-rpm reduction-gear motor. A special stage is mounted on the carriage block to hold a specimen of concrete for examination. The plane, finely ground surface of the concrete is viewed through a stationary stereoscopic microscope. One ocular is fitted with fine cross hairs so that the field of view is divided into four quadrants. Magnification is 30 to 54 diameters. Higher magnification can be obtained readily. For precise measurement of extremely minute voids, a petrographic microscope is used, measurements being made with a specially calibrated filar eyepiece. The carriage travels under the stereoscopic microscope when a clutch on the lead screw is engaged; translation of the carriage is indicated by a revolution counter geared to the lead screw. The stage is moved independently of the block by a manually operated gear train. A second counter operated by the gear train of the stage records the linear distance traveled by the stage with respect to the carriage block. The entire carriage block can be moved in a direction at right angles to the lathe bed by turning the cross-feed screw.

The concrete specimen is examined methodically along parallel equidistant traverses. These traverses are usually ¼ to ⅜ in. apart, but the spacing varies with the maximum size of the particles in the aggregate and surface area available for analysis. To begin the linear traverse, the clutch is engaged, and all aggregate and cement paste is traversed until the periphery of a void becomes aligned with the intersection of the cross hairs of the microscope. The clutch is then disengaged. The chord length across the void on the line of traverse is measured by manually operating the mechanism which moves the stage with respect to the carriage block so as to shift the opposite edge of the void to the intersection of the cross hairs. The clutch is engaged again and the traverse is continued until another void section is encountered on the line of traverse. The chord length is measured as before, and so on. Each void traversed by the intersection of the cross hairs is recorded on a counter so as to establish the total number of voids included in a determination. Thus, the linear traverse method indicates directly the total traverse length across the voids encountered, the total length of air-free concrete traversed, and the number of voids intersected.

From these data, several parameters of the void system are computed. The number of voids encountered per unit length of traverse n and the average chord length of the voids $\bar{l}$ are computed readily from the empirical data. A and α are computed as previously indicated.

PREPARATION OF GROUND SURFACES

The apparatus permits analysis of specimens as long as 18 in., but the dimensions of a specimen prepared for analysis are governed practically by the diameter of the grinding laps. The largest sample that can be prepared satisfactorily on a 12-in. diameter laps is about 6 x 5 in. Consequently, a sample prepared for analysis may comprise several slabs to insure adequate sampling.

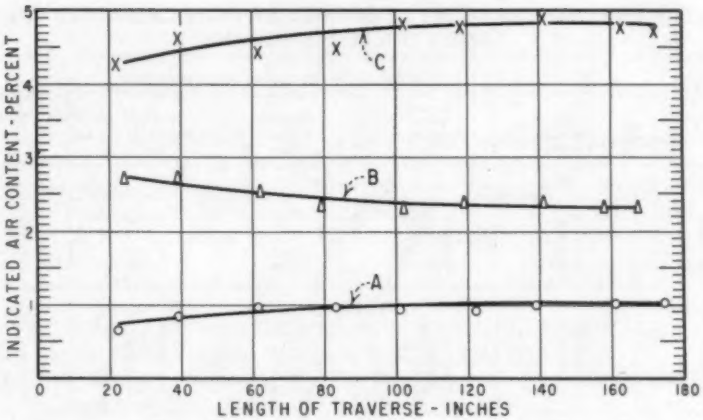

Fig. A-1—Change in indicated air void content of concrete with increasing length of linear traverse

After sawing of the slabs with a diamond blade, a smooth plane surface is obtained by grinding consecutively with No. 100 (to remove saw marks), No. 220, and No. 280 or equivalent abrasives on a rotating cast-iron lap and then finishing the grinding on a grooved lap using No. FFF and then No. 303½ or equivalent abrasives until the finish is suitable for microscopical examination. It is extremely important that the surface be washed and lightly scrubbed with a soft brush after each operation to remove grit and loose particles of concrete.

Very little difficulty was encountered in preparing surfaces of ordinary concrete that has been cured adequately. However, high air content, high water-cement ratio, lean mixes, and deterioration present problems in surface preparation. For example, at very high air content, the bubbles tend to coalesce, and the thin septa and ridges between the voids are easily lost during grinding. Void parameters determined on such a surface usually are only approximations because numerous chord interceptions must be estimated.

To minimize the difficulties of surface preparation, we have experimented with grinding procedures and with impregnation of the specimen to strengthen and support the near-surface concrete. Of all impregnating media tested, carnauba wax* is the most satisfactory. Briefly, the method is as follows: (1) heat the concrete specimen to 150 C; (2) melt the carnauba wax and brush on the surface of the hot concrete after it has been ground with coarse abrasives; (3) after the specimen has cooled nearly to room temperature, complete the grinding as outlined above; and (4) after a satisfactory surface has been prepared, remove the wax by heating the specimen (ground side up) at 150 C for 10 to 24 hr. During this time in the oven, the wax impregnating the surface is drawn by capillarity into the interior of the specimen so that the voids remain as well defined sections. Especially soft and fragile specimens may require a repetition of the impregnation procedure before a satisfactory surface is obtained.

ADEQUACY OF DATA

Brown and Pierson[12] have shown that a linear traverse of 100 in. should suffice to determine air content of concrete with reasonable accuracy. In our experience, this length of traverse is adequate to establish the air content of concrete containing aggregate as coarse as 1½-in. maximum size. The effect of the length of traverse on the indicated air content for three

*Carnauba wax is used as a base for commercial and household waxes and polishes. It usually can be obtained from manufacturers of these products or from chemical suppliers.

TABLE A-1—VOID PARAMETERS DETERMINED BY THREE OPERATORS ON THE SAME CONCRETE SPECIMENS*

Operator	Microscopically determined void parameters				
	Air content, percent	Average chord intercept, in.	Specific surface, in.$^{-1}$	Number of bubbles intercepted per in.	Spacing factor $\bar{L}$, in.
Specimen 1					
A	1.0	0.0127	315	0.787	0.0307
B	1.5	0.0150	267	1.007	0.0210
C	1.2	0.0134	299	0.926	0.0300
Average	1.23	0.0137	294	0.906	0.0272
Specimen 2					
A	5.0	0.0056	714	8.875	0.0066
B	4.9	0.0060	667	8.170	0.0072
C	5.2	0.0061	656	8.478	0.0071
Average	5.03	0.0059	679	8.507	0.0070
Specimen 3					
A	4.6	0.0033	1212	13,934	0.0039
B	4.7	0.0034	1176	13,850	0.0040
C	5.0	0.0035	1143	13,914	0.0040
Average	4.76	0.0034	1177	13,899	0.0040

*Length of traverse is approximately 50 in.

vibrated specimens containing ¾-in. maximum size aggregate is represented in Fig. A-1. The data show that the indicated air content changed very little after 100 in. of linear traverse were completed. For all three specimens, α was established adequately after a traverse of 50 in.

The length of traverse necessary to determine the air content with reasonable accuracy for concrete containing aggregate of 1½-in. maximum size or larger is primarily one of judgment. Some difficulty is usually encountered with 6-in. maximum size aggregate, and traverses of 250 in. or more may be required to accommodate irregular distribution of the large particles of aggregate. If the concrete exhibits an unusual degree of segregation, the total length of traverse is increased considerably and is conducted on a larger total surface when possible.

REPRODUCIBILITY

The results of the linear traverse are adequately reproducible provided the operator is trained properly. Especial attention must be given to instruction on distinguishing air voids from depressions in the ground surface produced by plucking of grains of fine aggregate. Care must be taken also in determining whether or not a void is tangential to the index mark or cross hair in the ocular. Difficulty may be encountered in identifying voids or, especially, in determining the position of the periphery, when the voids are partially or completely filled by deposits of crystals or gel.

Results obtained by three petrographers on the same specimens are summarized in Table A-1. The specimens were selected to represent a considerable range of air content and average bubble size. Specimen 1 is non-air-entrained concrete whereas Specimens 2 and 3 are air-entrained. For the air-entrained concrete, the determined parameters vary as much as 9 percent from the average, but the average deviation is only 2.3 percent. For the non-air-entrained concrete, the variation between operators is larger on the average, probably because of the sparse and erratic distribution of the voids.

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Note: References numbered consecutively beginning with Part 1 of this four-part series.

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37. Willis, T. F., Discussion of "Linear Traverse Technique for Measurement of Air in Hardened Concrete," ACI JOURNAL, Part 2, Dec. 1951, *Proc.* V. 47, pp. 124-1–124-2.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Bridges on German federal interregional highways (Brückenbauten an Bundesfernstrassen)

W. KLINGENBERG, *Der Bauingenieur* (Berlin), V. 32, No. 7, July 1957, pp. 245-258
Reviewed by ARON L. MIRSKY

In Germany, construction and maintenance of interregional highways are legally the responsibility of the federal government. Lengthy, well-illustrated article describes some of the many interesting bridges of major dimensions constructed on these highways in recent years. Structures are of all types but are grouped under two major headings: all steel with steel deck (protected by a wearing surface), and composite construction.

Construction Techniques

Concrete on the west coast of Jutland. Part 1—Concrete technique

G. M. IDORN, *Progress Report* B1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1958, 57 pp., 12 D.kr. (25 percent discount may be obtained by ordering full series)
AUTHOR'S SUMMARY

A survey of developments in concrete technique from about 1870 to 1952 used in the erection of numerous major structures of different types on the west coast of Jutland. Emphasis is given to the development in Denmark of special types of cement with respect to sulfate resistance. Information is given on (1) concrete materials including mixing water, aggregates, and admixtures; (2) mix proportions and recorded properties of fresh concrete; (3) different procedures of casting and curing. Some data on field testing of hardened concrete are presented.

The concluding remarks are few since the scope of the paper has been to outline the historical development rather than to evaluate any of the phases of technique described or the execution of any of the projects mentioned.

Design

Philosophy of structures

EDUARDO TORROJA (English version by J. J. POLIVKA and MILOS POLIVKA), University of California Press, Berkeley, 1958, 366 pp., $12.50

This book offers the reflections on structural design of one of the world's great engineers and architects—designer of the Esla railroad bridge, the famous Fronton Recoletos in Madrid, and many other elegant reinforced concrete structures. Systematic but non-mathematical, it discusses for both laymen and professionals the basic concepts, strategies, and ideals of advanced structural design as practiced in the modern world. Stress patterns, the nature of materials, economy, and other factors in design are analyzed. Domes, arches, spectacular thin-shell concrete roofs, massive dams, arch and truss bridges, aqueducts, dry docks, halls, lift-slab buildings, "hung" roofs, and many other structures are considered.

Functionalist in its emphasis, but without aridity, the volume will be thought-provoking for anyone seeking to understand the creative thinking that is transforming contemporary building techniques. The author concludes: "What is lacking is the popular education that will awaken the interest of those who are not

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engaged in this type of work. Also, there is need to stimulate in those who devote themselves to construction an awareness that if their work were better appreciated by laymen, it would be better for all, and especially for themselves."

Edge stresses in thin shells of revolution

W. H. Wittrick, *Report* SM-253, Aeronautical Research Laboratory, Melbourne, June 1957, 31 pp.
Applied Mechanics Reviews
July 1958 (Pao)

This paper reports on a method of satisfying all the edge conditions by superimposing on the membrane stresses an additional stress system which is governed by the full shell equations. It is known that, if the shell is thin, these additional stresses decay rapidly as the distance from the edge increases. This fact is exploited to obtain an approximate solution of the full shell equations for the additional stress system.

Static problem of concrete arch bridges (in Polish)

A. Brandt, *Inżynieria i Budownictwo* (Warsaw) No. 5, 1957, pp. 195-199
Polish Technical Abstracts
V. 29, No. 1, 1958

An analysis of arch bridges erected and of test results obtained concerning arch systems. The static characteristics of such structures are given as resulting from the fundamental feature—that of the cooperation of the arches with the bridge floor girders and connecting elements. Basic assumptions concerning the different methods of analyzing arch systems are also included.

Method of successive computation of internal forces acting in hyperboloidal water-cooling towers subject to wind pressure (in Polish)

J. Ledwon, *Inżynieria i Budownictwo* (Warsaw), No. 6, 1957, pp. 219-224
Polish Technical Abstracts
V. 29, No. 1, 1958

When computing internal forces acting in hyperboloidal water-cooling towers subject to wind pressure, load functions have been divided into three components. Since numerical computation of the components is laborious, the author advances a method of computing internal forces due to wind pressure, which does not involve breaking functions into harmonic components. Starting with the state of equilibrium of an elementary segment of a continually loaded symmetrical shell, the author obtains successive formulae for the internal forces. An analysis of the geometry of a single-shell hyperboloid is followed by a numerical example illustrating the application of the method discussed.

Materials

Insoluble residue determination in portland and portland-slag cements

W. J. Halstead and B. Chaiken, *ASTM Bulletin*, No. 229, Apr. 1958, pp. 60-65

The ASTM method of analysis for insoluble residue in cement was found to give variable results depending upon the analyst's interpretation of the specified conditions. The authors suggest modifications and clarifications to increase the reproducibility of the test results. They compared the federal method with the ASTM procedure and obtained different results for certain interpretations of the latter procedure.

Pressures developed in cement pastes and mortars by the alkali-aggregate reaction

R. G. Pike, *Bulletin* No. 171, Highway Research Board, 1958, pp. 34-36

Measurements with a special pressure cell, using an SR-4 strain gage as the sensitive element, show that the alkali-aggregate reaction in cement pastes and mortars may develop pressures in excess of 2000 psi.

Methods for rating concrete waterproofing materials

Ferruh Kocataskin and E. G. Swenson, *ASTM Bulletin*, No. 229, Apr. 1958, pp. 67-72

In an attempt to develop reliable test methods for evaluating the effectiveness of concrete waterproofers, it was found that a combination of saturated and unsaturated permeability tests is necessary, the latter covering more than one humidity condition. The materials tested showed to no advantage in the saturated permeability test. However, this test may have some merit where filler-type or pozzolanic admixtures are used. The unsaturated permeability test appeared to make

clear distinction between certain types of waterproofings and could be related to certain field situations. The capillary absorption test gave a good indication of the over-all effectiveness of waterproofings in cases where concrete is subjected to short periods of contact with moisture, with intermediate drying intervals.

The water repellent and bituminous admixtures showed up favorably in all the tests except the saturated permeability test. The calcium chloride admixture showed no beneficial effects in this study, apparently because the cement used did not respond to the usual accelerating action of calcium chloride on hydration.

The cement base paints provided varying degrees of benfit when painted on the side of the specimen subjected to wetting. The silicone application had a significant influence on moisture flow.

Method for accelerated testing of portland cements with regard to the development of their mechanical strength —report of the Swiss Institute of Material Testing and Research

Schweizer Archiv für angewandte Wissenschaft und Technik (Solothurn), V. 23, No. 3, 1957, pp. 65-70; No. 4, 1957, pp. 115-21; No. 5, 1957, pp. 146-56

CERAMIC ABSTRACTS
June 1958 (Hartenheim)

The aim of this investigation was to find a method of hydro-thermal treatment of portland cement giving results for compressive strength obtained in the autoclave that agree as closely as possible with the 28-day standard compressive strength. The test should require $\geq$ 48 hr.

The equipment used and the results obtained in six laboratories with different samples for setting behavior, volume stability, loss on ignition, content of free CaO and $Ca(OH)_2$, and fineness of particle are described in detail. The accelerated method has proved to be equiavlent to the 28-day standard test.

Portland pozzolan cements

TOYOFUJIMARU YOSHII and KINPE KANEKO, *Semento Gijutsu Nenpo*, V. 4, 1950, pp. 53-58

CERAMIC ABSTRACTS
June 1958 (Suzukawa)

The strength and permeability to air and water of pozzolan-lime mortar were determined; the floc test for durability was also made. No differences in the disappearance of alkalinity were observed between the mortar of portland pozzolan cement containing diatomaceous earth and that of portland cement. The mortars were stored in air for more than 6 months after removal from the mold.

Evaluation of the quick chemical test for alkali reactivity of concrete aggregate

RICHARD C. MIELENZ and E. J. BENTON, *Bulletin* No. 171, Highway Research Board, 1958, pp. 1-15

The following factors were investigated: (1) presence in the aggregate of minerals effecting extraneous reduction of alkalinity, including dolomite, antigorite, and calcite; (2) effect of agitation during the reaction period; (3) preliminary acid treatment of samples to remove carbonates; (4) measurement of pH during titration to determine the reduction of alkalinity; (5) modification of filtering procedures; and (6) variation in volume and concentration of the NaOH solution.

The findings do not justify modification of the test procedure at this time. However, it is recommended that a petrographic examination of the aggregate be performed as a guide to evaluation of the results of the chemical test.

Some properties of portland pozzolan cements

TOYOFUJIMARU YOSHII and SHIGEO AKAIWA, *Semento Gijutsu Nenpo*, V. 4, 1950, pp. 59-64

CERAMIC ABSTRACTS
June 1958 (Suzukawa)

Mortar strength at 3 days to 5 years, sulfate resistance, and heat of hydration of portland pozzolan cements are reported.

Miscellaneous observations on the alkali-aggregate reaction and the ionic charge on hydrated cement

ROBERT G. PIKE and DONALD HUBBARD, *Bulletin* No. 171, Highway Research Board, 1958, pp. 16-33

It has been demonstrated by an interferometric technique that calcium hydroxide has an inhibiting effect on the alkali-aggregate reaction, and it has also been demonstrated that opal may be caused to swell in the absence of any cement membrane. The fact

that aggregates are attacked more rapidly in a settling cement sludge than in the supernatant liquid has been explained by the higher pH values in the sludge. The magnitude and polarity of the excess ionic charge on various portland cements has been determined by a procedure which used the silver-ammonia-bromide complex as the indicator ion, and it appears that this ionic charge may be a contributing factor in the swelling and shrinking of hydrated cement.

Thiokol latex as applied to shells of reinforced concrete silos for petroleum by-products (in Polish)

A. Szurek and H. Owczarek, *Przeglad Budowlany* (Warsaw), No. 3, 1957, pp. 104-105
Polish Technical Abstrats V. 29, No. 1, 1958

Research has been carried on at the Institute of Building Technology over one of the polysulfide products—Thiokol A, in the form of latex. Experiments so far made have proved, among other things, that the quality of thiokol films and covers depends on the film-producing properties of latex, and that the vulcanization of latex contributes notably to improving the resistance of a thiokol film to chemicals.

Pavements

Model studies of prestressed rigid pavements for airfields

Paul F. Carlton and Ruth M. Behrmann, *Bulletin* No. 179, Highway Research Board, May 1958, pp. 32-50

Describes tests made on small prestressed gypsum cement slabs on an artificial subgrade of natural rubber. Conclusions are that small scale models of prestressed rigid pavements can be used to advantage in analysis and design for prestressed pavements.

Review of French and British procedures in the design of prestressed pavements

Phillip L. Melville, *Bulletin* No. 179, Highway Research Board, May 1958, pp. 1-12

Reviews current practice in the design of prestressed pavements for roads and airports in France and Britain. Concludes that current design practice results in overdesign for load carrying capacity in order to control deflection and furnish sufficient cover. Provides other tentative conclusions for the guidance of future design in this field.

Theoretical and practical aspects of prestressed concrete for highway pavements

Thomas Cholnoky, *Bulletin* No. 179, Highway Research Board, May 1958, pp. 13-31

Discusses theory of prestressed concrete pavement design and test data from the experimental slab at Patuxent by the Bureau of Yards and Docks, Department of the Navy.

Precast Concrete

Precast groins rebuild Florida's "lost" beaches

Concrete Products, V. 61, No. 6, June 1958, p. 30-31, 50

Describes an easily handled, easily assembled precast unit developed to break up wave action and collect sand, preventing storm and wind damage on shore property. The system employs flat precast units pierced by 7-in. hose which are joined by 6-in round posts.

Prestressed Concrete

Spinning molds form prestressed concrete lamp standards

R. Ironman, *Concrete Products*, V. 61, No. 6, June 1958, pp. 26-28, 52

Describes production methods used in Great Britain for spun concrete light poles. Includes pictures of various styles produced as well as detailed sections and a description of testing and production.

Prestressed concrete

P. B. Morice and E. H. Cooley, Pitman & Sons, Ltd., London, 1st Edition, 1958, 394 pp., 57s 6p

Discusses structural analysis and design of prestressed concrete and current construction and production practice in Great Britain. A brief section is devoted to the history, discussion of materials used, and development of theory. Separate chapters are then devoted to simple beams, continuous beams and frames, tanks and pipes, bridges, and advanced theory.

Other chapters include discussion of all important British systems, other European

systems covered in one chapter, pavements, and examples of both bridges and buildings. The book should be useful both as a text for undergraduate courses and as a reference source of information to practicing engineers unfamiliar with prestressed concrete.

Experiments on fire-resistance of prestressed concrete elements subject to compression (in Polish)

T. Klus, *Inzynieria i Budownictwo* (Warsaw), No. 9, 1957, pp. 308-318

Polish Technical Abstracts V. 29, No. 1, 1958

This paper discusses fire resistance tests made with small prestressed concrete elements 25 cm long, as well as with test pieces and cylinder specimens. The tests embraced problems of the effect of temperature on compressive strength and on elasticity coefficients. A description of the experiments is given in detail, together with results and analysis. The elements were tested at temperatures up to 600 C, kept at this level for 4 hr, and subsequently they were cooled rapidly. The protective concrete layer was 10 mm thick. Prestressed concrete elements under compression behaved much better in fire conditions than similar elements of concrete not prestressed.

Properties of Concrete

Early shrinkage characteristics of hand-placed concrete

B. W. Shacklock, *Magazine of Concrete Research* (London), V. 10, No. 28, Mar. 1958, pp. 3-12

Author's Summary

The shrinkage characteristics of fresh concrete from 10 to 15 min up to 2 or more days after mixing have been examined for a variety of mixes suitable for compaction by hand. The apparatus used for these tests, which is still in the prototype stage of design, is described briefly.

Under drying conditions, apparent expansion of the concrete, thought to be caused primarily by settlement of the specimen in the apparatus, occurs immediately after placing up to a time between 1½ and 4½ hr after mixing; by this time the concrete has stiffened sufficiently not to flow. Shrinkage, varying between 1×10^{-4} and 13×10^{-4}, takes place during the next 6 to 8 hr; there is then little further movement for a day or more. Concrete kept under water shows a greater apparent expansion, and no early shrinkage.

For mixes of approximately constant workability, amount of early shrinkage under drying conditions appears to increase as richness of the mix increases (with corresponding changes in the water-cement ratio), and it also increases as the amount of bleeding within the concrete decreases.

Studies on air-entrained concrete (in Japanese)

Shingo Seki, *Transactions*, Japan Society of Civil Engineers, No. 50, Dec. 1957, 54 pp.

Reviewed by Kiyoshi Okada

Process of strength development of concrete having larger or smaller water cement ratio is studied experimentally by measuring hydration heat and by microscopic observation. Concrete of smaller water-cement ratio shows the tendency of developing further strength at ages later than 91 days. Author also suggests that durability of concrete is substantially dependent upon the ratio of the freezable water to air content in concrete, and that the durability factor decreases linearly with increasing freezable water-air ratio.

Structural Research

Study of failure test in reinforced concrete beams (Contributo allo studio di trave in cemento armato in fase anelastica ed e rottura)

Ugo Rossetti, *Giornale del Genio Civile* (Rome), V. 95, No. 10, Oct. 1957, pp. 783-797

Reviewed by John I. Prelez

The article gives a very complete analysis of testing beams with different reinforcement coefficients and different patterns of bar spacing in the beam. The reader can also find a few interesting diagrams of failure test limits comparing the different designs of reinforcing steel.

Stresses in centrally loaded deep beams

P. H. Kaar, *Proceedings*, Society for Experimental Stress Analysis, V. 15, No. 1, pp. 77-84; reprinted as *Bulletin* D18, Portland Cement Association, July 1958

Outlines test procedures, results, and conclusions of studies on the action of stresses in

centrally loaded deep beams. Tests were not experimental in nature but were an illustrative study of the gradual transition from a simple linear flexural relationship to the highly nonlinear stress state characteristic of very deep beams. Several conclusions regarding action of deep beams were reached as a result, viz., there is considerable tension above and compression below the neutral axis; neutral axis often shifts its position with dimensional and loading changes; relative positions of loads and supports are of great importance; and stresses computed by ordinary flexure formulas are not accurate for these beams.

General

Impressive vibrations of towers and their elimination (Bemerkenswerte Schwingungen an Türmen und ihre Beseitigung)

J. Geiger, *Der Bauingenieur* (Berlin), V. 32, No. 7, July 1957, pp. 259-262

Reviewed by Aron L. Mirsky

Two examples are presented to illustrate value of policy of measuring, analyzing, and then prescribing, instead of arbitrarily strengthening. First example concerns a conveyor tower, where torsion from the electric motor drives was found to be the culprit. Second example is a church steeple; here large vibrations occurred during ringing of the heavy bells. Mathematical analysis, complicated by the highly irregular cross section of the steeple, and method of cure are described in some detail.

Proposed minimum standards for residential pools

National Swimming Pool Institute, Harvard, Ill., May 1958, 15 pp.

Presents minimum requirements considered consistent with health, safety, and public welfare for construction and design of residential swimming pools. The proposed standard is intended to assist regulatory agencies in developing new sanitary and building codes, and in revising older codes to cover installation and use of residential swimming pools. Prescribes minimum safety equipment, dimensions for diving, sanitary equipment, and structural requirements. The structural requirements are very general being largely performance requirements, and could be improved by reference to materials and design methods prescribed in existing codes or by establishing minimum safety factors.

Design and application of a simple acoustic strain gage

Civil Engineering and Public Works Review (London), V. 42, No. 618, Dec. 1957, pp. 1366-1370

Reviewed by J. J. Polivka

Frequency of transverse vibration of a stretched wire is proportional to the square root of the tension in the wire, on which law acoustic strain gages are based.

Author refers to various types of this very sensitive means of measuring strain; describes the design and operation of a simple inexpensive portable acoustic gage for measuring static strain by using the standard Maihak recording instrument, although any other recording instrument can easily be applied. Under normal conditions strains of the order 2×10^{-6} may be recorded. This gage is remarkably stable, and under normal conditions temperature variation has little effect on its precision. However, if subjected to rapid temperature fluctuation the gage must be properly shielded.

Estimating construction costs

R. L. Peurifoy, McGraw-Hill Book Co., Inc., New York, 2nd Edition, 1958, 446 pp., $10.75

An estimator's handbook suitable for use as a reference by construction engineers, design engineers, contractors, and architects. Describes in detail various types of estimates and methods of preparation. Individual chapters deal with various construction materials and many types of construction. Also includes coverage of workmen's compensation insurance, other insurance, and performance bonds. Tables giving production rates, especially those applying to labor, have been revised in this edition to give ranges in production rates instead of fixed rates.

New chapters or parts of chapters cover: additional equipment for and methods of drilling rock, stone masonry, frame residences, carpentry, interior trim and millwork, and glass and glazing. New material has been included on painting, lathing and plastering, roofing and sheet metal, plumbing, and electrical work.

ACI News Letter

Vol. 30, No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE October 1958

ACI headquarters entrance, p. 3

155 Reasons Why...

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How can durable aggregates be assured?

How do the requirements of placing affect the proportions of aggregates and cements?

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ACI

News Letter Contents

October 1958

On the Cover—Black terrazzo railing defines the stairway approach to the new ACI headquarters building slated for formal dedication during the 1958 regional meeting of the Institute in Detroit this month. Pneumatically applied concrete surface covers the electrical heating element which will free steps of winter snow and ice. A trio of articles by architect, engineer, and contractor, beginning on p. 417 of this month's JOURNAL, recounts some of the challenges in conception and execution of this unique structure.
—*Photograph by Hersey*

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'59 ACI SLATE

President—Phil M. Ferguson

Vice-president—Lewis H. Tuthill

The ACI Nominating Committee, headed by George W. Washa, University of Wisconsin, has announced the names of candidates to be submitted to member ballot at the 55th annual convention, Feb. 23-26, 1959, at the Statler Hilton Hotel, Los Angeles.

For President

Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, has been nominated to succeed Douglas McHenry as president for a 1-year term beginning at the February 1959 convention. Professor Ferguson is now completing his second year as Institute vice-president.

An ACI member since 1930, Professor Ferguson's achievements reveal his active participation in Institute affairs over a period of nearly 30 years. He has served on the Board of Direction, the Technical Activities Committee, and numerous technical and administrative committees. He is presently a member of ACI Committee 115, Research; Committee 208, Bond Stress; and Committee 318, Standard Building Code. He is ACI representative on the American Welding Society subcommittee on welding of bars, metal inserts, and connections in reinforced concrete construction. He has twice been recipient of the Wason Medal for Research: in 1957 for his paper "Some Implications of Recent Diagonal Tension Tests" (ACI JOURNAL, August 1956) and jointly in 1953 with J. Neils Thompson, University of Texas, for work reported in their paper, "Diagonal Tension in T-Beams Without Stirrups" (ACI JOURNAL, March 1953).

After graduating from the University of Texas in 1922 and receiving MS degrees from the Uni-

'59 ACI Nominees *continued*

versity of Texas in 1923 and the University of Wisconsin in 1924, he spent several years with Dwight P. Robinson and Co., New York, in structural design of power plants and industrial buildings and construction of hotels and apartment buildings.

Professor Ferguson returned to the University of Texas as associate professor in the field of structures and materials in 1928. He

Phil M. Ferguson

became professor in 1939 and was made chairman of the civil engineering department in 1943, a post from which he recently retired. Long recognized as an authority on structural design, Professor Ferguson has been consultant on a number of projects.

Past president of the Texas Society of Professional Engineers and active in a number of professional societies, he heads the ASCE Committee on Masonry and Reinforced Concrete.

For Vice-President

Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, has been nominated for a 2-year term as vice-president. Well known to JOURNAL readers, Mr. Tuthill has been a frequent contributor of technical papers for many years. Joining ACI in 1926, he has participated in nearly every phase of Institute activity. He has been a member of the Board

Lewis H. Tuthill

of Direction and now serves on the Technical Activities Committee and numerous ACI technical committees. He has prepared or collaborated on at least 14 papers for the JOURNAL. His paper, "Concrete Operations in the Concrete Ship Program" (ACI JOURNAL, January 1945) earned him the ACI Construction Practice Award. In 1956 he and William A. Cordon were awarded the Wason Medal for the most meritorious paper of the year for their joint contribution "Properties and Uses of Initially Retarded Concrete."

Mr. Tuthill assumed his present position in 1956; previously he had been associated with the Bureau of Reclamation in Denver for 17 years, and was chief of the Concrete Laboratory when he resigned to join the California State Department of Water Resources.

Since graduation from Oregon State College in 1920, Mr. Tuthill has been continuously engaged in design and construction work on irrigation, water supply projects, and the building of dams, with the technique and control of concrete as his primary interest.

Mr. Tuthill is currently chairman of ACI Committee 611, Inspection of Concrete, and Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete; he is a member of five other ACI technical committees: 116, Nomenclature; 207, Properties of Mass Concrete; 605, Hot Weather Concreting; 609, Compaction of Concrete by Mechanical Means; and 621, Aggregates.

Joe W. Kelly, professor and vice-chairman, Department of Civil Engineering, University of California, Berkeley, was elected to a 2-

year term as vice-president at the 1958 convention, and will continue in that office.

Board of Direction

Nominated to the four vacancies on the ACI Board of Direction for 3-year terms beginning at the February 1959 convention are: Roger H. Corbetta, George C. Ernst, E. A. Finney, and Walter J. McCoy.

Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York, has been a staunch supporter of ACI activities since joining in 1941.

Mr. Corbetta has been identified with precast concrete structures and prestressing methods since 1920. He has pioneered in the field of thin shell concrete construction and is one of the nation's leading concrete contractors.

A founder of the Concrete Industry Board of New York City, he served as its chairman for 5 years, and now is the Institute's representative on the board. Mr. Corbetta also is a member of Committee 324, Precast Reinforced Concrete, Thin Sections. He is a director of the National Concrete Contractors Association, and a member of the board of governors of the Building Congress of New York City.

George C. Ernst, professor of civil engineering and director of the Engineering Experiment Station at the University of Nebraska, Lincoln, graduated from the University of Michigan in 1925 and has been in academic work since 1930 at Iowa State College, University of Maryland, and University of Nebraska. In 1956 he resigned from the chairmanship of the civil engineering department at the University of Nebraska to devote full time to teaching and research.

Professor Ernst has been an ACI member since 1935, actively participating in administrative and technical committee work for a number of years. He is currently a member of Committee 115, Research, and will serve as chairman of the newly formed Committee 338, Torsion.

Professor Ernst has contributed a number of technical papers which have appeared in the JOURNAL, the latest being on "Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams," published in the October 1957 JOURNAL.

E. A. Finney, director of the research laboratory, Michigan State Highway Department, Lansing, is presently chairman of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports; a member of Committee 116, Nomenclature, and ACI representative to the Highway Research Board. He is also a member of the Technical Activities Committee.

Mr. Finney graduated from Allegheny College in 1923 and received his MS in highway engineering from Iowa State College in 1925. From 1925 to 1937 he served on the staff of the Civil Engineering Department and Engineering Experiment Station at Michigan State College and joined the Michigan State Highway Department in 1937. In 1942 he became director of the department's research laboratory. He has been especially active in research associated with air-entrained concrete, concrete pavement design and performance, and concrete materials.

Walter J. McCoy, director of research for Lehigh Portland Cement Co., Coplay, Pa., has been active in ACI since joining in 1946. He has already completed one 3-year term as a member of the Board of Direction, serves as chairman of the Technical Activities Committee, and is currently a member of Committee 115, Research; 116, Nomenclature;

1959 ACI CONVENTION

February 23-26—Los Angeles

Attention Exhibitors

The local committee for the 55th ACI Annual Convention in Los Angeles, Feb. 23-26, 1959, will sponsor an exhibit in conjunction with the convention.

Firms desiring to exhibit are requested to contact:

Glen Thomas
c/o Southern California Chapter
American Concrete Institute
Room 1058, 742 S. Hill St.
Los Angeles 14, Calif.

201, Durabiliy of Concrete in Service; 216, Fireproofing or Fire Protection of Structures; and 331, Structures of Concrete Masonry Units. He is also ACI representative to ASA Sectional Committees A-41 and A-42, on masonry and plastering, respectively.

Mr. McCoy graduated from Wisconsin State College and received his MS degree from the University of Minnesota. He is the author of numerous papers on analytical chemistry, portland cement, and concrete. In 1951 Mr. McCoy was awarded the ACI Wason Medal for the most noteworthy original research paper of the year, "New Approach to Inhibiting Alkali-Aggregate Expansion," which he co-authored with A. G. Caldwell.

Nominating committee

Members of the Nominating Committee who prepared the new slate of officers are, in addition to Chairman Washa; R. E. Copeland, National Concrete Masonry Association, Chicago; George C. Ernst, University of Nebraska, Lincoln; Ben C. Gerwick, Jr., of Ben C. Gerwick, Inc., San Francisco; Frank Kerekes, Michigan College of Mining and Technology, Houghton; Walter H. Price, Bureau of Reclamation, Denver; John P. Thompson, Portland Cement Association, Chicago; and Charles S. Whitney, Ammann and Whitney, Milwaukee and New York.

Twenty candidates have been chosen for the 1959 Nominating Committee, five of whom will be elected at the February convention.

Candidates are:

ARTHUR R. ANDERSON
DELMAR L. BLOEM
S. D. BURKS
R. A. BURMEISTER
WILLIAM A. CORDON
WALDEMAR C. HANSEN
S. B. HELMS
THOMAS B. KENNEDY
GEORGE E. LARGE
JAMES A. McCARTHY
GEORGE H. NELSON
GENE M. NORDBY
H. F. PECKWORTH
T. C. POWERS
CHESTER P. SIESS

Northern California members contemplate local ACI chapter

Preliminary steps to organize a local ACI chapter in northern California have been taken by a sponsoring committee of members in that area. S. D. Burks of Dewey and Almy Chemical Co., San Leandro, is acting as chairman of the group; Joe W. Kelly, University of California, Berkeley, is chairman of a subcommittee appointed to develop a statement of objectives for the proposed chapter and formulate by-laws. He is being assisted by Al Klein and Milos Polivka, both of the University of California.

Harry Thomas, Pacific Gas and Electric Laboratory, Emeryville, is chairman of the membership subcommittee with Ed Howard, Pacific Cement and Aggregates, Inc.; Max Neumann of Woodward, Clyde and Associates, Oakland; and Lewis H. Tuthill, California State Department of Water Resources, Sacramento, as committee members.

Other members of the sponsoring committee are: Orville Jack, Permanente Cement Co., Cupertino; Ben C. Gerwick, Jr., of Ben C. Gerwick, Inc., San Francisco; Carl Rollins, Basalt Rock Co., Inc., Napa; Frank Killinger, Hales Testing Labs, Oakland; Dick Allen, Dewey and Almy Chemical Co., San Leandro; and Jay Jellick, Portland Cement Association, San Francisco.

The sponsoring committee is anxious to acquaint all ACI members in the northern California area with the current status of the proposed chapter. The necessary signatures have been secured to accompany the petition to Institute headquarters requesting formation of this chapter. Application is now pending and will be acted upon at the October meeting of the Board of Direction during the 1958 regional meeting in Detroit.

New slag testing lab established

National Slag Association and Standard Slag Co. have opened a new research and testing laboratory at Canfield, Ohio, with the purpose of enhancing the application of slag in the concrete industry for the future. The facility has four main testing centers: moist curing room, concrete laboratory, freeze-thaw room, and aggregate laboratory.

six new ACI technical committees approved

Constantly expanding the scope of ACI activities, the Board of Direction has approved the formation of six new technical committees bringing the total number of ACI technical committees in operation up to 53. Combined footings is the subject assigned to one of the new committees; another will study the strength evaluation of existing concrete structures. Two of the newly formed committees will concentrate on torsion, and allowable stresses in reinforcement, respectively. The other two committees will deal with portland cement plaster and low-pressure steam curing practices.

Committee 336 studies footings

Paul Rogers, consulting engineer, Chicago, has been appointed chairman of Committee 336, Combined Footings. This committee will collect records of successful designs, compare theoretical assumptions of soil behavior, and compare design methods in use and/or those prescribed by other codes. All types of combined footings now outside the scope of the ACI Building Code will be considered. Generally accepted methods of design presented in various reference books, authorized by various statutory codes both domestic and foreign, and those in use by practicing design offices will be considered. In particular, performance records will be correlated with design methods in developing recommendations.

Strength of existing structures topic for Committee 337

Carlos D. Bullock of Burgwin and Martin, Topeka, Kan., has assumed chairmanship of Committee 337, Strength Evaluation of Existing Concrete Structures. The committee will collect information on methods of evaluating capacity of existing concrete structures. Reports will include recommendations for a theoretical analysis, field data required, and methods of securing such data.

Recommendations by this committee for procedures of structural analysis will include only those which differ from procedures accepted in design such as may be required for evaluating effects of fire, weathering, abrasion, chemical attack, shock, settlement, cracks, frozen or faulty joints, or openings cut and additional work added after completion of construction. Recommendations will include suggestions for tests appropriate to determine strength of concrete in place and effectiveness of reinforcement. Suggestions will be included for methods to evaluate properties such as strength, pulse velocity, surface hardness, abrasions resistance, sonic modulus, permeability, and creep.

1958 REGIONAL MEETING

Statler-Hilton Hotel, Detroit

October 27-29

Full coverage in December issue

Torsion to be considered by Committee 338

George C. Ernst, University of Nebraska, Lincoln, is chairman of Committee 338, Torsion, which will study available research and existing requirements for design and statutory codes. The committee will report on recommendations for allowable torsional stresses, singly and in combination with shear, flexural, or axial stresses for plain and reinforced concrete. Recommended methods for analysis and design for elastic or ultimate strength methods will be given.

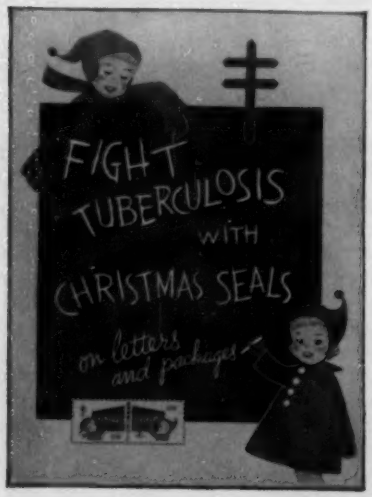
Committee work will cover design of plain, reinforced, or prestressed concrete for torsion,

special reinforcement for torsion, combinations of spiral, hoop, and tie reinforcement with longitudinal reinforcement. Studies of existing codes may include the Greek Building Code, British Code, and Indian Railway Code. Studies of research will include results of elastic and plastic theory compared to strength tests.

Committee 339 to work on reinforcement stresses

Another of the recently approved committees, Committee 339, Allowable Stresses in Reinforcement, is headed by Robert Sailer, U. S. Bureau of Reclamation, Denver. This committee will evaluate successful practice and research rather than initiate research. Its objectives will be an evaluation of performance and a comparison of safety factors in practice rather than development recommendations for safety factors.

The committee will analyze available published research data, foreign practice, and available examples in this country utilizing high strength reinforcement in nonprestressed concrete. The group will then prepare a summary of successful practice in which allowable steel stress is related to strength of reinforcement including limitations which were imposed upon, and prescribed methods of estimating crack width and deflection.



Committee 624 to write plaster specification

Joseph D. McNulty, Sr., of McNulty Brothers, Chicago, is chairman of Committee 624, Portland Cement Plaster. This committee is assigned the task of preparing a specification for portland cement plastering. The group will study the recommendations of the former committee, other recommendations, research, and present practices. Recommendations will be developed for interior and exterior application, reinforcement or backing required, decorative effects, jointing, lightweight aggregate, admixtures, machine and manual application, and cements suitable for such plaster.

Steam curing guide will be developed by Committee 717

J. J. Shideler, Portland Cement Association, Chicago, heads up Committee 717, Practice in Low-Pressure Steam Curing. This committee will start work on a comprehensive guide to the best practice for utilizing low pressure steam and high temperatures in curing concrete. Recommended practices will be prepared for accelerated curing of structural concrete elements and common types of precast concrete products. Recommendations will be based upon present practice and presently available research data. Research data collected and recommendations developed by other ACI committees will be considered for incorporation in the report.

Recommendations of this committee will eventually include use of steam at atmospheric pressure; desirable temperatures, and timing and rate of heating; effect of admixtures; effect of various cements; and procedures for normal, lightweight aggregate, and foam concretes. Recommendations suitable for a range of applications including concrete masonry, structural precast elements, precast prestressed elements, and pipe will be developed.

Harris establishes European office

Frederic R. Harris, Inc., consulting engineers, New York, N. Y., has established a subsidiary in The Hague, Netherlands, to serve clients in Europe and the Middle East.

Architect: Milton M. Schwartz & Associates, Chicago; Structural Engineer: Miller Engineering Co., Chicago

2,100 tons of RAIL STEEL reinforcing bars

assure strength and safety for

nation's tallest reinforced concrete structure

Rail steel provides extra strength and safety to EXECUTIVE HOUSE—40-stories of concrete nearly complete in Chicago—the tallest reinforced concrete building in the U.S. High-strength rail steel reinforcing bars were specified and used throughout, not only in the building's framework of columns, four shear walls and 57 caissons 118′ below ground level, but also for an extra strength-safety factor in the flat beam and tile joist floors and cantilever slabs. By specifying re-bars made of Rail Steel, these designers have made ample provision for unpredictable overloads and insured maximum shock resistance.

There are other important benefits too when considering high-tensile rail steel reinforcing bars for your next design. One is the uniformity resulting from the rigid specifications under which rail steel's raw material has been produced. Another reason is the prompt and accurate service you get from the network of strategically located manufacturers. Remember, rail steel reinforcing bars meet both ASTM specifications A-16 and A-305. For more information, contact

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UNIQUE STRUCTU RA

Employs

Ultimate strength design

High strength reinforcement

A CONCRETE MAN'S concrete laboratory began operation recently at Portland Cement Association Research and Development Laboratories, Skokie, Ill. Designed in its own "element," the laboratory, billed as the world's largest testing machine, is itself the result of the most recent developments in the concrete field. An unencumbered 56 x 120-ft test floor is the main testing apparatus.

Floor design

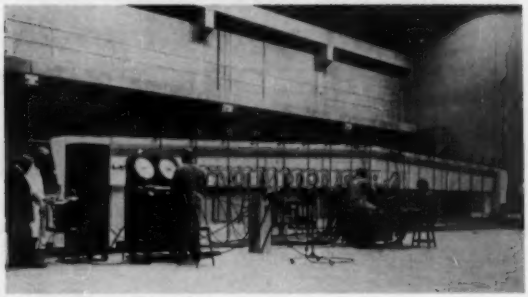
The floor was designed, using bridge design methods, to act as a hollow concrete box girder in the longitudinal direction and a Vierendeel girder in the transverse direction. It is the first floor of its type to be designed according to principles of reinforced concrete; with only 2 percent of alloy steel deformed bars as main reinforcing, it functions completely as reinforced concrete. The specially rolled bars meet requirements of ASTM A 305 and have a yield strength of 70,000 psi.

Test loads are applied against specimens through steel tie rods which project through 2½-in. holes in the 2-ft "upper chord" of the cast-in-place floor and are stressed by hydraulic jacks which pull against the underside. The service capacity of the floor is about 10,000,000 lb, and full sized girders, roof slabs, or shells can be tested to destruction. All structural elements of the building were designed by the ultimate strength method with concrete specified to have a minimum 28-day compressive strength of 5000 psi.

Above the test floor the building is built entirely of precast elements. Columns, 2 ft square, 42 ft 11 in. long, are cantilevered from reinforced sockets in the floor to form bents 22 ft on centers. Wall panels are precast lightweight expanded shale concrete cast on the test floor, tilted up, and fastened with clip angles to the columns. The panel concrete was designed for a 28-day strength of 4000 psi.

Tests reported

Roof girders fit into slots in the columns and span 58 ft. The joint, similar to a mortise and tenon, provides practically no resisting moment. The girders were designed according to the appendix to "Building Code Requirements for Reinforced Concrete (ACI 318-56)" with certain modifications and extensions. Tests were made to substantiate the departures. (See p. 469 of this month's JOURNAL for a report of these tests.) Columns



U RAL LABORATORY

Tilt up construction

Concrete sculpture

Precast constuction

are tied together in the longitudinal direction with Z-shaped spandrel beams which also provide a 3-ft roof overhang.

20-ton crane supported by girders

The 20-ton crane which serves the laboratory is supported by precast girders reinforced with high strength steel. Negative reinforcement over the supports was lapped and welded to provide continuity. The girders were designed with a higher safety factor than other structural elements to reflect the high safety factor customary in the design of the crane itself.

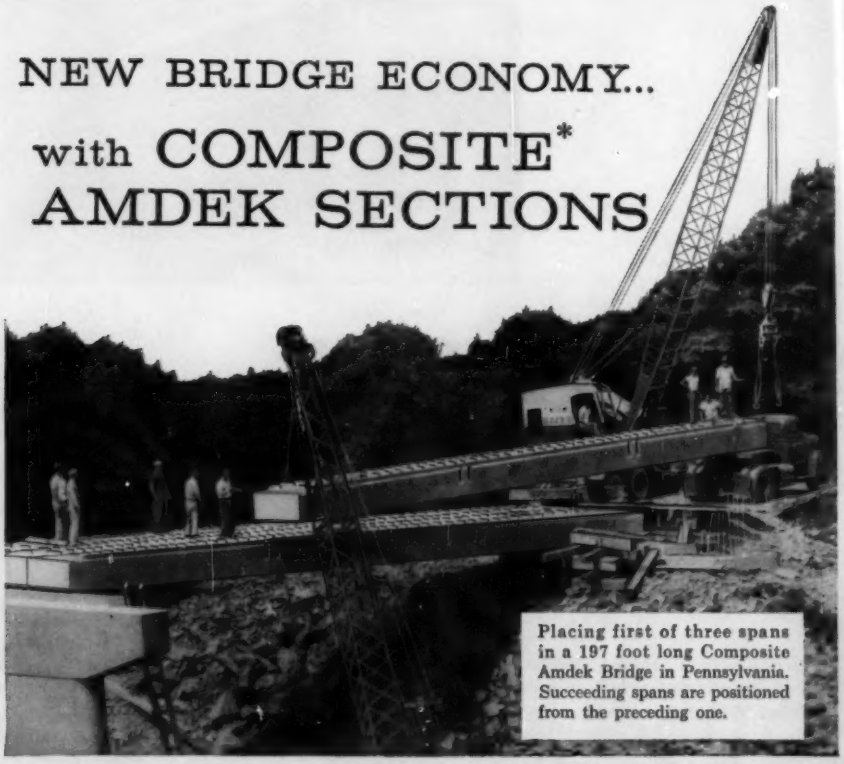
A 4 ft 8 in. wide balcony is provided along one wall for visitors. Brackets supporting the balcony are attached to the columns by two post-tensioned high strength steel bolts. The roof deck is composed of hollow precast concrete units spanning 20 ft between roof girders. Negative reinforcement was installed over the girders to give some continuity. Insulation and built-up roofing were applied to the concrete units.

The columns, cast with smooth surfaces, contrast with the 20 x 30-ft wall panels which were cast against textured rubber form liners. This exterior texture, which has a vertical emphasis, is highlighted by small square indentations arranged in a pattern of squares. The inside surface of the panels is broomed.

Above the wall panels in the testing area are 5-ft precast panels which are ornamented with designs expressing various aspects of testing. The originals were modeled in clay and plaster molds made from them, in which the concrete was cast. One sculptured panel is centered in each bay and flanked by panels of vertical fluting. Windows replace the sculptured panels in an office section of the building. The main entrance foyer is enclosed with large precast sculptured wall panels produced in the same manner as the panels used in the upper portion of the wall. These panels are 8 ft wide and 11 ft high.

The Structural Laboratory is under the direction of Eivind Hognestad, manager, development section, and supervised over-all by Douglas McHenry, director of development. Architects were Dunlap and Esgar; and general contractor G. A. Fuller Co., Chicago

NEW BRIDGE ECONOMY...
with COMPOSITE* AMDEK SECTIONS



Placing first of three spans in a 197 foot long Composite Amdek Bridge in Pennsylvania. Succeeding spans are positioned from the preceding one.

Another example of American-Marietta PROGRESS in CONCRETE

*After the new four-foot wide Composite Amdek Sections are positioned, a four-inch concrete deck is cast over the raised stirrups. The box sections and surfacing then become an integral working unit.

Composite Amdek Sections with concrete surfacing require less concrete, less handling and no form work—which results in considerable savings. Independent dynamic tests have proven their tremendous fatigue strength.

For information on this and on standard three-foot wide Amdek Beams without raised stirrups for use under asphalt surfacing, contact American-Marietta Company.

AMERICAN-MARIETTA COMPANY
CONCRETE PRODUCTS DIVISION
GENERAL OFFICES:
AMERICAN-MARIETTA BUILDING
101 EAST ONTARIO STREET, CHICAGO 11, ILLINOIS, PHONE: WHITEHALL 4-5600

Positions and Projects

New Jersey construction bureau uses ACI standards

The Bureau of Construction of the State of New Jersey recently adopted a standard specification for prestressed and precast concrete construction based on recommendations as published in the report of ACI-ASCE Joint Committee 323, "Tentative Recommendations for Prestressed Concrete" (ACI JOURNAL, Jan. 1958). The specification stipulates that bidder is to abide by the provisions of the ACI Building Code (ACI 318-56).

Other ACI standards incorporated by reference in the specification are: "Recommended Practice for Winter Concreting (ACI 604-56)"; "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)"; and "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)."

Rowe awarded grant to study curved structures

Robert S. Rowe, professor and chairman of the civil engineering department at Duke University, has received a U. S. Army Officer of Ordnance research grant of $10,952 to study stresses and strains in doubly curved structures.

Before going to Duke in 1956, Professor Rowe was associate professor of civil engineering and director of the rivers and harbors section at Princeton.

Professor Rowe has been an ACI member since 1952 and is currently a member of Committee 322, Permissible Stresses for the Design of Unreinforced Concrete.

Special Libraries Center inaugurates translation service

The Special Libraries Association Translation Center at the John Crerar Library, Chicago, has announced that it can now furnish, on a subscription basis, printed catalog cards for current scientific and technical material which has been translated into English from all languages, including Russian. Thus it makes available in easy-to-use card form the thousands of citations given in the center's bibliographical journal, *Translation Monthly*.

Four types of subscriptions to translation catalog cards are available: full coverage of *Translation Monthly* (approximately 12,000 titles yearly); coverage of all translations currently received by the center (approximately 6750 titles yearly); coverage of all Russian translations received by the center (approximately 3100 titles yearly); and coverage of all titles in specific subject fields.

Further information may be obtained from: SLA Translation Center, John Crerar Library, 86 Randolph St., Chicago.

Cosgrove appointed to Maryland plant

George V. Cosgrove has been named superintendent of the new Alpha Portland Cement Co., plant at Lime Kiln, Md. Mr. Cosgrove, who started with Alpha in July of 1951 as plant engineer at Manheim, W. Va., was combustion engineer for all Alpha plants before moving to his present position.

Cornell names building in honor of Upson

Upson Hall, named in tribute to Maxwell M. Upson, a graduate of the class of 1899 and a trustee of the University since 1925, was formally presented to Cornell University on June 14 at a special dedication ceremony the Cornell campus in Ithaca, N. Y.

The newest and largest of the eight buildings comprising the university's new Engineering Quadrangle, Upson Hall will accommodate approximately 700 undergraduate and graduate mechanical engineering students. The reinforced concrete structure contains approximately 126,000 sq ft of floor space.

Mr. Upson has been an active ACI member since 1912. He was president of the Institute in 1926, has served on ACI's Board of Direction and technical committees, and has contributed several technical papers for JOURNAL publication.

Now 70% of all concrete highways are

Steel reinforcement in portland cement concrete is now required for highway construction in 23 states. These 23 states account for more than 70% of all highway construction!

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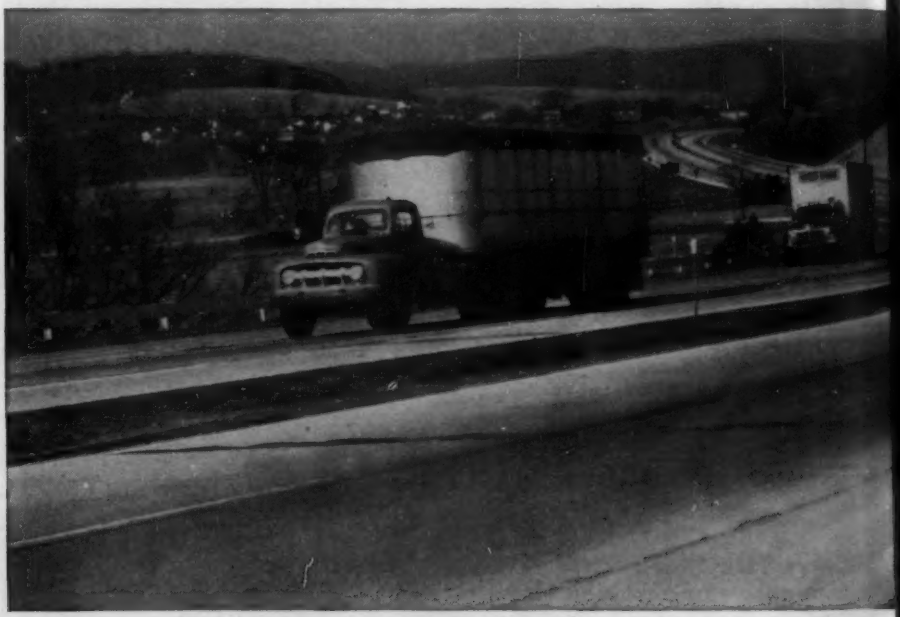

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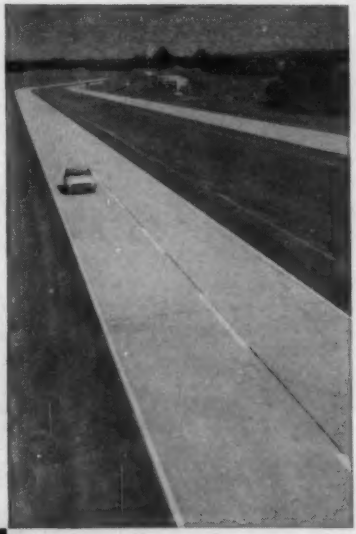

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Concrete products industry surveyed

Employing approximately 70,700 persons in 5090 plants, the concrete products industry attained an estimated volume in 1957 of $920 million in value of products shipped, according to the second annual Survey of Industrial Buying Power published by *Sales Management* magazine.

It was disclosed that Los Angeles County, Cal., is the nation's leading concrete products county, employing approximately 2400 persons in 85 plants.

The nation's top 20 concrete products counties are shown by the survey to contain 618 plants, representing 12.1 percent of the industry total.

Simpson awarded MIT degree

The Massachusetts Institute of Technology has conferred the DSc degree in civil engineering on Prof. Howard Simpson. Dr. Simpson also was honored recently by the Boston Society of Civil Engineers with the award of the Desmond Fitzgerald Medal. This medal, the society's senior award, was given for the paper, "The New ACI Code—Its Implications and Ramifications."

An ACI member since 1947, Dr. Simpson currently serves on ACI Committee 318, Standard Building Code and ACI-ASCE Committee 323, Prestressed Reinforced Concrete. He is a member of the firm Simpson, Gumpertz and Heger, Inc., Cambridge, Mass.

Prior named technical service manager by Dewey and Almy

Melville E. Prior has been appointed technical service manager of the construction chemicals department, Dewey and Almy Chemical Division, W. R. Grace and Co., Cambridge, Mass.

Mr. Prior, with 25 years experience in cement and concrete technology, will specialize in technical assistance and service to distributors and customers, development of technical literature, and the company's participation in the work of the construction industry's professional and technical societies.

Mr. Prior is chairman of Subcommittee 1 of ACI Committee 212, Admixtures, and a member of Committee 115, Research. He is also a member of the Highway Research Board, and active in a number of technical committees of the American Society for Testing Materials.

Teng joins Chicago engineering firm

Wayne Teng, formerly of the faculty of the Illinois Institute of Technology, has joined John F. Meissner Engineers, Inc., Chicago, as head of the structural department.

Dr. Teng has had extensive experience in the structural engineering field. Formerly assistant chief structural engineer for Skidmore, Owings, and Merrill, he has worked on a number of outstanding buildings in the Chicago area. He was in charge of the design of several structures for the new Air Force Academy at Colorado Springs, Colo. While associated with Sargent and Lundy he participated in the design of a number of steam and atomic power plants.

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Prestressed Concrete Institute holds fourth annual convention

The 1958 annual meeting and convention of the Prestressed Concrete Institute was held September 21-25 at the Edgewater Beach Hotel, Chicago. In addition to a comprehensive technical program, the convention featured an exhibit of new equipment and materials for the prestressing industry.

The opening session was presided over by PCI President Ben C. Gerwick, Jr. with Charles Luckman of Pereira and Luckman,

Los Angeles, featured as keynote speaker of the convention.

Two panel discussions were devoted to marketing and promotion with Robert Singer of Ben C. Gerwick, Inc., San Francisco, acting as chairman and moderator. ACI member, Peter J. Verna, Jr., Concrete Materials Co., Charlotte, N. C., served as moderator at the panel discussion on yard handling, transportation, and erection techniques; Joe Schneider of Edward Campbell Co., Vineland, N. J., conducted the panel discussion on quality concrete for prestressing.

Maury Bender, Knoerle, Graef, Bender and Associates, Inc., Baltimore, moderated the panel discussion, "Draping Strands at Illinois Toll Road and Elsewhere;" ACI member Jack Janney, consulting engineer, Chicago, led a panel discussing interpretation of specifications in relation to inspection.

Charles C. Zollman of Schupack and Zollman presided at the morning session on September 25 when a report was presented on the proposed PCI certification of prestressing plants. A panel discussion followed on long span prestressed bridges with Morris M. Schupack of Schupack and Zollman acting as moderator.

At the closing session of the convention Ben C. Gerwick, Jr. reported on the recent Berlin conference on prestressed concrete. Prof. T. Y. Lin of the University of California reported on prestressing in the Soviet Union and compared it with United States practices.

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concrete houses cement company

The two-story, 34,000-sq ft structure, built by Riverside Cement Co. at Crestmore, Calif., exemplifies wide use of manufactured concrete products, and the trend toward attractive suburban centers for industrial enterprises in southern California.

One of the wings houses extensive laboratory facilities on the ground floor, and large drafting and engineering areas and offices on the second floor. The other wing has a ground floor garage with conference room and supervisory offices on the second floor. The two wings are connected on the second level by a precast post-tensioned balcony slab.

Among some of the new materials and innovations utilized were lightweight masonry concrete block and an ornamental girder canopy with cast-in-place design.

Other features include concrete floors, columns, and roof framing in the structural shell, floating spiral staircases (exterior and interior), precast sun-shade units, varying forms of multicolored concrete panels and tiles, and patios with pebble aggregate paving. Designed by Allison and Rible, architects; constructed by Twaits-Wittenberg Co., Los Angeles constructors-engineers.

FLOATING SPIRAL STAIRWAY at Riverside Cement Co.'s recently completed office and laboratory building at Crestmore, Calif., is only one of many eye-catching architectural features

Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

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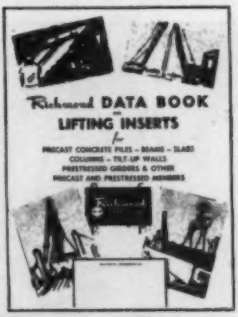
No matter what type of precast units are involved, Richmond can supply exactly the right type of insert for the specific job.

These units are designed with adequate extra strength and for simple operation in lifting and bracing precast concrete components.

Send for your copy of the Richmond Data Book on Lifting Inserts giving complete technical data, dimensions, working loads and ultimate strengths in various strengths of concrete. At the same time ask for your copy of the latest Richmond Handbook showing the complete line of form tying anchorage and accessory devices for concrete construction backed by 47 years in this field.



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This unusual monolithic prestressed beam and slab design resulted from alternate bids permitted by Philadelphia's progressive Department of Streets* . . . Savings of more than 20% over conventional steel design were passed on to the taxpayers.

Each 120 ft. long span, 34 ft. wide; including two traffic lanes, sidewalk, and half of the center median strip was concreted in one continuous 260-yard operation . . . Since 8-10 hours elapsed during placement, Plastiment was specified to control initial set. Plastiment provided these additional benefits: better compaction, greater density, and faster development of stressing strength.

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AD 36-7

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Westerhoff named professional engineers' society director

Russell P. Westerhoff, a vice-president of Ford, Bacon and Davis, Inc., New York, has been named to a 2-year term as a director of the National Society of Professional Engineers to represent the New Jersey Society of Professional Engineers.

Mr. Westerhoff's professional services have ranged from industrial economic reports to plant design and construction. He has been with Ford, Bacon and Davis since 1928, when he received his MS in civil engineering from the Massachusetts Institute of Technology.

He is an ACI member and active in a number of other professional societies.

Billig accepts London position

Kurt Billig has accepted the position of chief engineer for the development of nuclear power stations with Taylor Woodrow Construction, Ltd., London. Prior to this appointment Dr. Billig had been with the United Nations Technical Assistance Administration as consultant to the government of Pakistan.

Ideal Cement uses prestressed channels in unique conveying system

A contract for the longest permanent cross-country transport belt conveying system ever constructed, 5½ miles in length, was recently awarded by Ideal Cement Co. of Denver to Link-Belt Co., Chicago.

The entire multimillion dollar system comprises seven conveyors arranged consecutively to provide continuous flow of material and will transport crushed limestone and shale at a rate of 1000 tons per hr from Ideal's Lawrence, Okla. quarry to its Ada, Okla. cement mill. The length of the longest individual conveyor will be 11,920 ft.

One of the most unusual features of the cross-country belt conveyor will be the use of prestressed lightweight concrete channels, which both support the conveyor and provide a continuous all-weather cover throughout its entire length. The prestressed concrete channels and precast concrete piers will be furnished by the Thomas Concrete Pipe Co. of Ada. Construction has begun and completion is scheduled for early 1959.

The Ada plant expansion is part of a $170 million company-wide expansion program which is designed to increase Ideal's productive capacity to 40 million barrels annually by 1965.

Ultimate strength design seminar in St. Louis on November 1

An ultimate strength design seminar will be held all day, November 1, on the campus of St. Louis University, St. Louis, Mo.

Panel participants include: Eivind Hognestad, Portland Cement Association, Chicago; John McCarthy, head of the Civil Engineering Department, St. Louis University; James Cronin, professor of civil engineering at St. Louis University; Calvin Garrett, Scott and Garrett, Oklahoma City; W. L. Fraley from PCA in Chicago; and Arnold Bock, PCA, St. Louis.

Killgore elected president of Florida concrete group

Ernest S. Killgore, Pinellas Concrete Product Co., St. Petersburg, Fla., was elected president of the Florida Concrete and Products Association at the recent annual convention of the group in Ft. Lauderdale.

Schupack and Zollman open civil engineering office

Charles C. Zollman, head of Charles C. Zollman and Associates consulting engineers, Newtown Square, Pa., has announced the formation of Schupack and Zollman for the practice of civil engineering, particularly in precast and prestressed concrete. Morris Schupack was formerly chief engineer and vice-president of the Preload Co., and will continue to serve as consultant to that firm.

The new engineering firm will concentrate in the field of long span prestressed structures. Charles C. Zollman and Associates will continue to serve the prestressed concrete producer in the design of pretensioning plants and plant manufactured products.

Both Mr. Zollman and Mr. Schupack are ACI members and are currently serving on ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

TECHKOTE LIFT SLAB COMPOUNDS

"Multi-story, Lift Slab Job"

300 Y-S 400 Y-S 500 Y-S

Techkote Company developed the original curing and bond-breaking agent used successfully during the early testing phases of the Youtz-Slick Lift Slab Method of Construction. This compound is known as *Techkote Lift Slab Compound 300 Y-S.* As additional requirements for special treatments became apparent, Techkote Company developed *Techkote Lift Slab Compound 400 Y-S* and *Techkote Lift Slab Compound 500 Y-S.*

It is highly essential in casting these large slabs that maximum strength concrete be produced. Four basic requirements to obtain this are: correct mix-design, thorough mixing, workmanlike placing and proper curing. Techkote Lift Slab Compounds produce this proper curing. They comply with leading Governmental Specifications.

TECHKOTE LIFT SLAB COMPOUND 300 Y-S *Wax-base compound. Provides excellent cure and bond-breaking at low cost. Is effective on all surfaces including floated, broomed, etc.*

TECHKOTE LIFT SLAB COMPOUND 400 Y-S *Wax-free compound. Produces a tough film that provides excellent cure and bond-breaking and is resistant to rainfall and abrasion. Film is compatible with good quality adhesives, paints, etc.*

TECHKOTE LIFT SLAB COMPOUND 500 Y-S *Wax-free compound. Leaves a minimum film that provides good cure and bond-breaking. Film weathers away quickly. Requires a minimum of preparation for painting, placing tile, etc.*

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Who's Who This Month

Minoru Yamasaki

Reflections on concrete and its use in the distinctive design originated for the new Institute building are offered by the architect. Minoru Yamasaki in "ACI Headquarters Presented a Challenge in Concrete," p. 419.

Mr. Yamasaki received his BS in architecture from the University of Washington, Seattle, then did graduate work at New York University where he was an instructor of water color in 1935-36. Volunteering his services to help with competition design for the Oregon State Capital at the office of Githens and Keally, New York, he found himself working as an architect when the firm won the competition.

During the depression and following years he studied, taught, and worked for various architectural and design firms, gradually building respect and winning the admiration of architects throughout the country. Then in 1945 he became design chief at Smith, Hinchman and Grylls in Detroit. His reputation as an outstanding architect continued to rise as he worked on various projects.

In 1949 he formed a partnership and the firm of Leinweber, Yamasaki, and Hellmuth was organized with headquarters in Detroit and St. Louis. This firm served as the architects for the Lambert-St. Louis Air Terminal; the American Consulate, Kobe, Japan; and the John J. Cochran Garden Apartments for the St. Louis Housing Authority. Among the schools the firm designed, two have received special awards.

Since 1955 Mr. Yamasaki has been carrying on his work as a principal of Yamasaki, Leinweber and Associates, architects and engineers, Birmingham, Mich. In the past 5 years his philosophy and expression of design have won numerous awards and definitely have established him as one of America's outstanding architects. Three Detroit area buildings recently designed by the firm are: the McGregor Memorial Conference Center, the Reynolds Metal Co. sales office, and the classroom and administration building for the Detroit Arts and Crafts Society. The laboratory and warehouse for Parke-Davis at Menlo Park, Calif., was also a recent design by the firm.

The ACI headquarters building was an award winner for Mr. Yamasaki, taking the top award in the commerce division in the 1957 *Progressive Architecture* composition.

Charles S. Whitney

Past ACI president Charles S. Whitney, structural engineer for the Institute headquarters building, conveys some of the problems he considered in structural design of the unique folded plate roof in his article entitled "Cantilevered Folded Plate Roofs ACI Headquarters," p. 427.

Mr. Whitney, a partner in the engineering firm of Ammann and Whitney, New York and Milwaukee, scarcely needs introduction to JOURNAL readers. He is well known for his contributions to reinforced concrete design, particularly in shell construction and other arched structures and in ultimate load design. A graduate of Cornell University, since 1922 he has been in private practice as a designing and consulting engineer.

Active in affairs of the Institute since 1920, he twice received the Wason Medal, and in 1951 was accorded the Alfred E. Lindau Award "in recognition of his many contributions to reinforced concrete design practice." Mr. Whitney was president of the Institute in 1955 and has served on numerous ACI administrative and technical committees. He is currently chairman of ACI-ASCE 326, Shear and Diagonal Tension; and a member of ACI Committee 214, Evaluation of Results of Compression Tests of Field Concrete; Committee 312, Plain and Reinforced Concrete Arches; Committee 318, Standard Building Code.

Mr. Whitney has twice been awarded the Fuertes Graduate Medal of Cornell University and also has received the J. James R. Croes Medal from the American Society of Civil Engineers.

John R. Strang

Transforming the approved plans for the ACI headquarters building into a tangible structure was directed by John R. Strang, vice-president of Pulte-Strang, Inc., Ferndate, Mich. Mr. Strang unfolds a story of the actual work on the building in his article "Construction for ACI," p. 431, highlighting some unusual and difficult problems encountered.

Mr. Strang graduated from the University of Minnesota in 1941. Following service in the Corps of Engineers during World War II, he was associated with Barton-Malow, Oak Park, Mich., for 5 years.

Since the formation of Pulte-Strang, Inc., in 1953, the firm has constructed an impressive list of noteworthy projects in the Detroit area including the Liberal Arts Building at the University of Detroit; the Quick Auditorium at Eastern Michigan College, Ypsilanti; Bloomfield Junior High in Bloomfield Hills, Mich., and is now engaged in construction of the Dearborn, Mich., Youth Center.

Mr. Strang is also a principal in the recently formed Foamcrete, Inc., which will manufacture and distribute Betocel foamed concrete in Michigan.

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Lyndon Welch

"Folded Plate Dome Ideal for Auditorium" on p. 441 was written by Lyndon Welch, chief structural engineer, Eberle M. Smith Associates, Inc., Detroit.

Mr. Welch received his BA from Harvard in 1943 and his MS from Massachusetts Institute of Technology in 1948. He served in the United States Navy from 1943-46.

During 1949-50, Mr. Welch was an instructor in the School of Architecture and Design, University of Michigan. After a period of employment in architectural and engineering offices, he joined Eberle M. Smith Associates as chief structural engineer in 1954, directing design and erection of lift slab, prestressed girders, cylindrical shells, and folded plates.

Felix J. Samuely

Felix J. Samuely, consulting engineer, London, reports on his wide experience with "Folded Slab Construction" on p. 447. Mr. Samuely was born in Vienna and educated in Berlin. He has been in practice as a consulting engineer in England for 25 years.

Always interested in current developments, in recent years he has given most of his time to two problems: the development of skin structures made from various building materials such as reinforced and prestressed concrete, steelwork, timber, and aluminum, and wide span floors in prestressed concrete.

Among buildings for which Mr. Samuely has acted as structural engineer are technical colleges, schools, blocks of flats, offices, factories, and hospitals in various parts of the United Kingdom. His latest buildings of special interest include the new United States Embassy in London, a church in Stamford,

Conn., constructed entirely in precast concrete, the British pavilions at the 1958 Brussels Universal and International Exhibition, London County Council blocks of flats of 14, 18, and 25 stories, and the development of St. Thomas's Hospital area. He has been particularly interested in the architectural expression of engineering structures.

Otto Safir

"Reinforced Concrete Core Main Structural Element in 22-Story Office Tower" on p. 461 was written by Otto Safir, consulting engineer, Vancouver, B. C. After graduation in engineering in 1931, Mr. Safir did postgraduate work in Switzerland on reinforced concrete bridges. Then until 1945 he worked with consulting engineers and designing contractors in England and had his own consulting practice there from 1945-49.

Mr. Safir's main interests have been structural development and he has been responsible for the design of several reinforced concrete bridges. In 1945 he designed one of the first precast structures in England, a double bay shed with two 75-ft spans. He was also associated with various precast concrete housing and factory projects in England. Since moving to Canada in 1949 he has engaged in similar work there, notably in British Columbia.

J. R. Gaston and Eivind Hognestad

J. R. Gaston and Eivind Hognestad are joint authors of "Precast Concrete Girders Reinforced with High Strength Deformed Bars," on p. 469 of this issue.

Mr. Gaston is an associate development engineer in the Structural Development Section of the Skokie, Ill., laboratories of the Portland Cement Association. At the present he is conducting development work on precast concrete components and the joints between these components.

Prior to joining the Portland Cement Association, he graduated from Rose Polytechnic Institute in 1950 and obtained his MS from the University of Illinois in 1953. During the interim between his university work and joining PCA, Mr. Gaston spent two years

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in the U. S. Army Ordnance Corps. He has been an ACI member since 1950.

Mr. Hognestad, manager, Structural Development Section, Portland Cement Association, Chicago, is an author well known to ACI JOURNAL readers. Prior to joining the PCA staff in 1953, Dr. Hognestad was research associate professor of theoretical and applied mechanics at the University of Illinois.

He obtained his MS degree in civil engineering from the University of Illinois and the degrees of civil engineer and doctor of technical sciences from the Norweigian Institute of Technology. In 1947 he received a research appointment to the University of Illinois where he worked on various projects pertaining to concrete.

Mr. Hognestad has been an ACI member since 1947. He is vice-chairman of ACI-ASCE Committee 328, Limit Design, and a member of ACI Committee 318, Standard Building Code and ACI-ASCE Committee 326, Shear and Diagonal Tension.

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A. C. Scordelis, T. Y. Lin, and H. R. May

A trio of authors, A. C. Scordelis, T. Y. Lin, and H. R. May have collaborated in preparing "Shearing Strength of Prestressed Concrete Lift Slabs," p. 485.

A. C. Scordelis is an associate professor of civil engineering at the University of California, Berkeley. After having served in the U. S. Army Corps of Engineers during World War II, Professor Scordelis received a BS from the University of California in 1948 and an MS from the Massachusetts Institute of Technology in 1949. He has had experience in structural design through his work for the Pacific Gas and Electric Co., Bechtel Corp., and Ernest J. Kump, architect, at various times since 1948. He has been a member of the University of California faculty since 1949 and has participated in teaching and research in the fields of structural theory, reinforced concrete, and prestressed concrete.

A member of ACI, ASCE, and ASEE, Professor Scordelis also serves on ACI-ASCE Committee 712, Recommended Practice for Pre-Cast Structural Concrete Design and Construction; ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs; and ACI Committee 335, Deflection of Concrete Building Structures. He and Professor Lin are currently collaborating in research on the behavior of continuous concrete slabs prestressed in two directions.

T. Y. Lin, after graduating from Chiaotung University, China, received his MS in civil engineering at the University of California in 1933. Returning to China he worked with the government railways, becoming chief of design and planning for the Yunnan Burma Railway. In 1946 he joined the faculty of the University of California.

An ACI member since 1950, Professor Lin is a member of ACI Committee 314, Rigid Frames for Buildings and Bridges and ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

Last year Professor Lin was general chairman for the World Conference on Prestressed Concrete held in San Francisco. He recently returned from the USSR heading an American delegation observing Soviet work in prestressed concrete. In addition to teaching and research at Berkeley, he is a partner in the Los

Angeles firm of T. Y. Lin and Associates, consulting engineers specializing in prestressed concrete.

Howard R. May, engineer with the firm of T. Y. Lin and Associates, graduated from the University of California in 1951. He then worked two years with the Alaska Road Commission as a highway engineer. The following two years were spent with the U. S. Army Corps of Engineers.

Mr. May returned to the University of California at Berkeley in 1955 and spent one year doing graduate study in structures and a year as a research engineer studying prestressed concrete. He joined ACI in 1957.

Richard C. Mielenz, Vladimir E. Wolkodoff, James E. Backstrom, and Richard W. Burrows

Messrs. Mielenz, Wolkodoff, Backstrom, and Burrows present the fourth part of a four-part paper on "Origin, Evolution, and Effects of the Air Void System in Concrete" on p. 507.

Part 4 is entitled "The Air Void System in Job Concrete." Part 1 covering "entrained air in unhardened concrete" appeared in the July 1958 ACI JOURNAL; Part 2, "Influence of Type and Amount of Air-Entraining Agent," was published in the August issue; Part 3, "Some Factors Influencing Size and Spacing of Air Voids in Concrete and Their Effect on Freezing and Thawing Durability" appeared in the September issue.

Biographical sketches of the authors appeared in the July and August issues of the JOURNAL.

Dorrance appointed Turner technical advisor

Nelson L. Doe, vice-president, Turner Construction Co., New York, announces the appointment of George P. Dorrance as technical advisor for the company.

Brown organizes engineering firm

Francis L. Brown formerly a partner in the firm of Brown and Blauvelt, New York, N. Y., has announced the formation of a new consulting engineering firm, Brown Engineers, also with offices in New York.

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Honor Roll

January 1—September 30, 1958

The contest between Blas Lamberti and Joseph J. Waddell is very evident this month with only 1 credit separating the two men. Influencing others to join ACI gives each member the satisfaction of knowing he has helped in furthering the technical knowledge of concrete and concrete construction.

Blas Lamberti	14
Joseph J. Waddell	13
Phil M. Ferguson	10
James A. McCarthy	7
L. M. Legatski	6
Ernst Maag	6
H. C. Pfannkuche	6
Jerome M. Raphael	6
J. P. Thompson	6
Samuel Hobbs	5
Carl H. Koontz	5
Douglas McHenry	5
C. P. Siess	5
Ernest L. Spencer	5
Moises Bendahan	4
Celso A. Carbonell	4
Jaime de las Casas	4
P. E. Ellen	4
N. J. Everard	4
Frank E. Legg, Jr.	4
John B. Porter	4
Walter H. Price	4
C. H. Scholer	4
Clarence A. Walkwitz	4
Luther E. Bell	3
Vincent R. Cartelli	3
Carl E. Ekberg, Jr.	3
Aleck S. Evans	3
Richard W. Gunn	3
Martin J. Gutzwiller	3
Paul A. Hansen	3
Robert B. Harris	3
A. T. Hersey	3
E. L. Howard	3
Kenneth M. Huber	3
E. Richard Kummerle	3
Daniel S. Ling	3
George A. Mansfield	3
H. P. Mittet	3
E. Schmid	3
Howard Simpson	3
Otis D. Small	3
O. C. Struthers	3
Ellis S. Vieser	3
Byron P. Weintz	3
William M. Avery	2
Robert B. Banning	2
Nicandro Jose Barboza	2
O. R. Bell	2
Massimiliano R. Berretti	2
Carlos D. Bullock	2
Miles N. Clair	2
W. L. Collier	2
W. S. Cottingham	2
Lancelot A. Fekete	2
Louis A. Gottheil	2
Einar Block Hansen	2
Arch Hull	2
H. Alan Johnson	2
Thomas C. Kavanagh	2
F. R. Khan	2
R. G. King	2
Carl O. Knop	2
E. Vernon Konkel	2
George Kurio	2
James R. Libby	2
Ciceron Hiedra Lopez	2
William McGuire	2
Bryant Mather	2
Frank B. May	2
Charles F. Moran	2
James A. Murray	2
Wendell H. Nedderman	2
Neftali Penaloza R.	2
Abdur Rahman S. Rasul	2
Clarence Rawhauser	2
Herbert Rusk	2
Francesco Sardella	2
Leo L. Schaut	2
George Shervington	2
George B. Southworth	2
Donald L. Strange-Boston	2
Ferruh Taskin	2
T. W. Thomas	2
Oscar J. Vago	2
Ruben Cano Vicario	2
Jose Antonio Vila	2
Ivan A. Villamil	2
M. R. Vinayaka	2
Charles S. Whitney	2
Tsu-ming Yang	2

New Members

The Board of Direction approved 67 Individual applications, 3 Corporations, 21 Juniors, and 3 Students, making a total of 94 new members. Considering losses due to resignations, deaths, and nonpayment of dues the total membership on Sept. 1, 1958, was 9596.

Individual

AINSWORTH, JAMES F., Beaumont, Texas (Branch Mgr., Southwestern Labs.)

BENDERSON, NATHAN, Troy, N. Y. (Constr. Mtls. Engr., Benderson Laboratory)

BOWERS, HENRY KEITH, North Battleford, Sask., Canada (P. E., Mgr., Forrester & Scott, Archs.)

BRADSHAW, HUGH A., Oklahoma City, Okla. (Chf. Engr., Standard Testing & Engrg. Co.)

BRAMANTE, RODOLFO P., Buenos Aires, Argentina (Engr. in Chge. Struct. Section, R. y E. Minvielle)

CAMPBELL, RONALD F., Villawood, N. S. W., Australia (Mgr., control technical & business operations, Monier Prestressed Concrete Pty. Ltd.)

CANALI, FEDERICO, Milano, Italy (Designer & Contr. of prestressed R/C structures)

CIPPARRONE, ACHILLE, Roma, Italy (Engr., Struct. Dept., Litchfield, Whiting & Bowne, Archs. & Engrs.)

COCHRAN, ROBERT E., Peoria, Ill. (Assoc., C. C. Briggs, Arch.)

LOOKING AHEAD

Oct. 20-24, 1958—National Safety Congress and Exposition, Conrad Hilton and other hotels, Chicago, Ill.

Oct. 23-25, 1958—National Society of Professional Engineers, Fall Meeting, St. Francis Hotel, San Francisco, Calif.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hilton Hotel, Detroit, Mich.

Nov. 1, 1958—Ultimate Strength Design Seminar, St. Louis University, St. Louis, Mo.

Nov. 12, 1958—Cleveland Engineering Society, 1958 Construction Conference, Cleveland Engineering and Scientific Center, Cleveland, Ohio

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

Dec. 7-10, 1958—American Institute of Chemical Engineers, Annual Meeting, Netherland Plaza Hotel, Cincinnati, Ohio

Dec. 10-11, 1958—National Construction Industry Conference, Fourth Annual Meeting, Sherman Hotel, Chicago, Ill.

Jan. 12-15, 1959—39th Annual National Concrete Masonry Association Convention and 11th Concrete Industries Exposition, Cleveland Public Auditorium, Cleveland, Ohio

Feb. 23-27, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.

COHEN, P., Johannesburg, South Africa (Struct. Engr. Design of R/C Structures, Stewart, Sviridov & Oliver)
COOPER, LESLIE TREVOR, Parkstone Poole, Dorset, England (Works Mgr., Sharp, Jones & Co., Ltd.)
CRARY, JAMES F., Manhattan, Kansas (Asst. Prof. of Applied Mechanics, Kansas State College, Engr. Concrete Constr., Kansas & Wilson & Co., Engrs.)
DEAN, WARREN E., Rolla, Mo. (Bldg. Contr.)
DOMLESKY, L. A., Harrisburg, Pa. (Proj. Engr., Gannett, Fleming, Corddry & Carpenter, Inc.)
DUBOIS, PAUL E., Oklahoma City, Okla. (Chf. C. E., Coston-Frankfort-Short, Archs. & Engrs.)
EASTERDAY, ROBERT E., St. Louis, Mo. (C. E., St. Louis Water Dept.)
EVANS, JAMES A., Massillon, Ohio (Cons. Engr., Head of James A. Evans & Assocs.)
FARGO, LEWIS F., Riverside, N. J. (Engrg. Dir., Research & Control Lab. Div., Trap Rock Industries, Inc.)
FERGUSON, M. W., Roanoke, Va. (Formerly owned and operated Pre-Shrunk Masonry Sales Corp.)
FLUGGE, W., Stanford, Calif. (Prof. of Engrg. Mechanics, Stanford Univ.)
FOGLI, PETE, San Mateo, Calif. (H. E. Casey Co.)
FORTOUL PADRON, DR. EDDY, Caracas, Venezuela (C. E.)
FOWLER, E. W., New York, N. Y. (Director of Codes & Standards, National Board of Fire Underwriters)
HARTER, JOHN W., Trenton, N. J. (Lab. Mgr., United Testing Lab.)
HAYES, WESLEY T., Oakland, Calif. (Partner, Graham & Hayes, Struct. Engrs.)
HEATH, C. O., Corvallis, Oregon (Assoc. Prof. of Mech. Engrg., Oregon State College)
HOFFMASTER, BOB, Long Beach, Calif. (Chf. Harbor Engr., Port of Long Beach)
IMMENDORF, DAVID N., Easton, Pa. (Field Engr., Alpha Portland Cement Co.)
JEWELL, LLOYD V., Detroit, Mich. (Asst. Div. Engr., Surveying & Insp., Detroit Edison Co.)
JURKOVICH, W. J., Sacramento, Calif. (Sr. Bridge Engr., State of Calif., Div. of Hwys.)
KIMMELL, DWIGHT, Las Vegas, Nev. (Owner & Mgr., Superior Concrete Specialties)
KUHN, CHARLES J., Charleston, W. Va. (Pres., Kuhn Constr. Co.)
LEE, CHESMAN A., Evanston, Ill. (Design Draftsman, U. S. Steel Corp.)
LIM, CHIU-CHING, San Nicolas, Manila, Philippines (C. E., John Gotamco & Sons, Inc.)
LOEWER, WILLIAM A., Chicago, Ill. (Housing & Cement Prod. Engr., PCA)
MANRIQUE, C., FRANCISCO, Bogota, Colombia (Preload de Colombia, Ltda.)
MICKELSON, A. J., Ft. William, Ont., Canada (Struct. Engrg. & Arch., Mickelson, Fraser & Haywood)
MIHALKO, WILLIAM, Johnson City, N. Y. (Field Engr., Alpha Portland Cement Co.)
MILBURN, KEITH BENJAMIN CHARLES, Villawood, N. S. W., Australia (Mng. Dir., Concrete Industries (Australia) Ltd.)
MORGAN, W. B., Houston, Tex. (Partner, Lloyd & Morgan, Archs.)
MURRIA, JUAN, Zulia, Venezuela (Engr., Compania Shell de Venezuela)
ODMAN, DR. SVEN T. A., Stockholm, Sweden (Tech. Dir., Cement-och Betonginstitutat)
ORE, ELWOOD L., Denver, Colo. (Matls. Engr., USBR)
OROZCO, ENRIQUE LOZANO, Culiacan, Sin., Mexico (Branch Mgr., Casas y Obras, S. A.)
PALUMBO, PETER S., Jr., Pt. Pleasant, N. J. (Jr. Design Engr., Porter, Urquhart, McCreary & O'Brien)
PERALTA, ALFREDO, Caracas, Venezuela (Vice-Pres., Industrial Incorporada, C. A.—"ININCA")
PLACE, ANDREW S., South Bend, Ind. (Home Builder)
POLISNER, LEE, Boston, Mass. (Sales Mgr., Engrg. of Steam Equipment, Burkhart Engrg. Assocs.)
RAMIREZ, I. PERES, Caracas, Venezuela (Engr. Insp. of Hwy., Ministry of Public Works)
RAVELO MENESES, JAVIER, Santiago de Cuba, Cuba (Practicing Arch., Plans, Bldgs., Const.)
REESBY, RAYMOND G., Galveston, Tex. (Chf. Engr., Struct. Cons. Engr.)
RODA, LEONCIA, Lima, Peru (C. E., Ministerio de Fomento y Obras Publicas)
RUSSO, JORGE ZEGARRA, Miraflores, Lima, Peru (Tech. Mgr., Jobs Dept., Asst. Dir., Concrete Prod., Pre and Poststressed, Precast Floors, Lime & Gypsum Prod., Guillermo Payet S. A.)

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SLOAN, E. S., San Francisco, Calif. (Sr. Engr., Design, Bechtel Corp.)
STAMPER, H. N., Houston, Tex. (Chf. C. E., Humble Oil & Refining Co.)
SUCRE, ALFREDO ESCOBAR, Caracas, Venezuela (Oficina Tecnica de Ingeniera—especialidad en Calculos Estructurales)
TRAPANI, SAM, Springfield, Ill. (Dir., Trapani Engrg. Co.)
TROUTMAN, I. VOYT, Lyndhurst, Ohio (Mgr., Cleveland Div. Sales, Master Builders Co.)
VAN SUETENDAEL, ARTHUR J., III, New York, N. Y. (Design Engr. & Office Mgr., A. Carl Stelling Assocs.)
VARGA, JOSEPH E., Birmingham, Mich. (Asst. Prof. of Arch., Univ. of Detroit)
VESCIA, ALDOUS, Jedda, Saudi Arabia (Prof. Engr.)
VOLK, JOHN C., JR., Wilmington, Del. (Gen. Field Engr., PCA)
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CRIMP, ARTHUR ELLIOTT, Neutral Bay, N. S. W., Australia (Instructor, N. S. W. Univ. of Tech.)
EVERINGHAM, GEORGE LEROY, JR., Navesink, N. J. (Matls., Eng., Field Work, Porter, Urquhart, McCreary & O'Brien)
FITZGERALD, ROBERT W., Worcester, Mass. (Engrg. Instructor, Worcester Junior College)
GRANGER, GEORGE A., JR., Angola, Ind. (Instructor, Tri-State College)
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LAU, WINSTON KWOK-CHU, Chicago, Ill. (Struct. Designer, Skidmore, Owings & Merrill)
LONTOS, DEMETRIOS K., Corpus Christi, Tex. (Asst. Engr., Texas State Hwy. Dept.)
MARSHALL, BENJAMIN FRANKLIN, III, Pleasantville, Pa. (Engr. in Trng., Frick & Sweeney)
MOODY, PAUL, London, Ont., Canada (Struct. Design Engr., M. M. Dillon & Co., Ltd.)
MULLER, JOHN A., Ozone Park, N. Y. (Designer, Severud-Elstad-Krueger Assocs.)
NUYENS AVILA, RENE, Guatemala, Guatemala (Constr. Supt., Dirrecion General de Caminos)
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WARRINER, PHILIP C., San Jose, Calif. (Asst. Bridge Engr., State of Calif., Div. of Hwys., Bridge Dept.)
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Student

ALAMI, ZUHEIR Y., Austin, Tex. (Univ. of Tex.)
MATHEWS, WILLIAM R., Windsor, Ont., Canada
MENG, YUEN KAI, Melbourne, Australia (Univ. of Melbourne)

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

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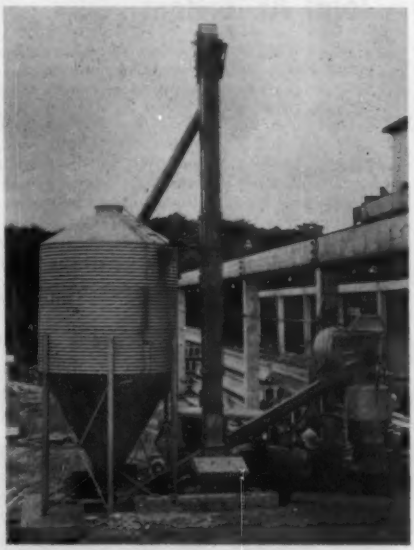
Especially suited for small and medium size ready-mixed concrete plants and all types of concrete products



plants, the company states that low cost and high production are the keynotes for the design of these bulk cement plants.

Heavy galvanized coating is said to assure many years of trouble-free operation. Complete set of aeration fittings are supplied to condition the cement for proper flow. Available in various combinations with elevators, screw conveyors, and manual or automatic batchers.—Engineered Equipment, Inc., Waterloo, Iowa

Curing membrane for concrete

Crete-Cure, a white pigmented concrete curing membrane said to have high dispersion characteristics has been developed by Thiem Products, Inc.

In explaining the advantages of this new curing membrane, a spokesman for the firm stated that materials used in the manufacture of such membranes have had a tendency to settle or separate; in Crete-Cure, the compounded particles of materials are given greater suspension resulting in a heat reflecting concrete curing agent that reportedly offers the maximum in moisture

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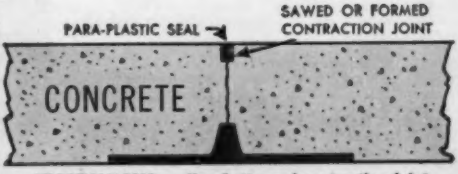
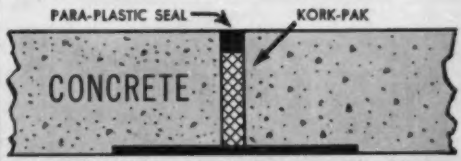


"BASEAL" 5309 sealing bottom of contraction joint



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retention and a more uniform consistency which flows freely and is less apt to clog or contaminate spraying equipment. Manufacturer states a successful test program on Wisconsin highways was conducted with Crete-Cure during 1957. The membrane is custom mixed on order to meet the specifications of each state highway department.—Thiem Products, Inc., 9800 West Rogers St., Milwaukee 19, Wisc.

Undertrack screw unloader

Latest addition to the Andrews Machine Co. line of screw conveying equipment is the firm's new Model P-52 undertrack screw unloader.

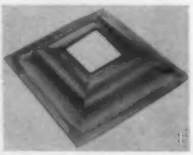
Designed especially for semipermanent installation and use, this model is buried under the rails of a siding at the selected unloading point and can be installed horizontally or inclined.

Construction and design is basically the same as the P-51 overtrack screw unloader. Standard length of unit is 5 ft 9 in. but is available in any length required, as measured from the center line of the tracks to the point of desired discharge, or tie-in with main conveying equipment.

Power units may be either electric or gasoline engine. Manufacturer states that one man can readily handle operation of the unit which provides efficient, high-capacity unloading for cement, gypsum, potash, and most free flowing materials.—Andrews Machine Co., 359 E. Main St., Decatur, Ill.

Vinyl plastic waterstops

A critical part of the design and construction of the Cambridge Electron Accelerator, an alternating gradient synchroton which is a joint project of Massachusetts Institute of Technology and Harvard University, is the use of specially fabricated soft vinyl plastic waterstops which surround the concrete pads on which magnetic accelerators rest. All the pads are below the ground water table and therefore subject to water infiltration.



The pads, set independently in a common concrete floor are surrounded on four sides by a 1-in. expansion joint. Within this joint are the special Flextrip crimped waterstops, installed 10 in. above the bottom of the pads. The waterstops must seal the pads against water seepage from below, and yet they must transmit the absolute minimum of stress to the pads that results from expansion and contraction of the surrounding concrete. In the expansion joint, above and below the waterstop, glass wool insulation is placed and sealed on the surface with an asphalt rubber sealer. Thus, each pad, with its magnetic accelerator, is protected against stress that would cause an adverse affect on the accelerating process.—Water Seals, Inc., 9 S. Clinton St., Chicago 6, Ill.

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SOLVAY Calcium Chloride speeds but does not change the normal chemical action of portland cement. Impartial tests by the National Bureau of Standards proved its advantages in cold weather concreting. This use of calcium chloride is recommended or approved by leading authorities, including American Concrete Institute and Portland Cement Association.

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SOLVAY dealers and branch offices are located in major centers from coast to coast.

Coating for precast concrete

Armor-Crete, a recently introduced flooring resurfacing coating, has found a new market in the precast field where many manufacturers have adapted the material to resurfacing panels of precast concrete, according to the developer of the product. Armor-Crete may be applied over the exterior of a concrete building as easily as it can be used to make an acid-proof floor according to the manufacturer. They recommend it for resurfacing any precast unit which poses difficult protective or decorative problems as well as in areas where hard abrasive wear will be encountered.

Various color and textures can be provided from brilliant tones to subdued shades. The coating is troweled over the unit in a thin topping which cures quickly. Material is supplied in two kit sizes with the container used as the mixing vessel for each kit.—Preco Chemical Corp., 415 Lexington Ave., New York, N. Y.

Road roughness indicator

The Soiltest Model CT-444 road roughness indicator is basically styled after the Bureau of Public Roads design which was developed to provide standardizable

equipment for measuring road surface roughness. The manufacturer reports, however, that many modifications and new features have been incorporated in recent models.

The complete unit consists of a test trailer, ramp, electronic control and recording unit, and a panel instrument truck. The test trailer is towed by a specially modified panel truck at a constant speed (usually 20 mph). Variations on the pavement profile cause the smooth, sensitive test tire to raise and lower as it rolls along the pavement. This vertical movement is converted into electronic pulses which are relayed to the instrument panel in the truck. At the same time an oscillograph recorder plots a permanent record of the pavement profile. Additional data available in Bulletin CT-444.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

36-hp concrete saw

The Clipper Manufacturing Co. announces a new 36-hp concrete saw, Model C-363, for high production contraction joint cutting on highways, turnpikes, airfields, and for heavy duty trenching operations.

This model features one-piece frame design for precision weight distribution and easy maneuverability. Ball bearing positive screw feed quickly raises and

lowers the blade into the cut and provides accurate cutting depth control.

According to the manufacturer the C-363 is built to withstand the toughest cutting requirements, yet weighs 135 lb less than its predecessor. Six reinforced V-belts deliver 100 power to the blade.

Other features cited by the manufacturer that contribute to greater ease in handling and more productive job performance include: improved method of tightening the V-belts, new depth indicator, premium Wisconsin engine with Stellite valves and valve rotators, and water pump with built-in clutch for disengagement.—Clipper Manufacturing Co., Suite 185, 2800 Warwick, Kansas City, Mo.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested direcly from the manufacturers listed below.

CYCLAP BULLETIN 482—Describes laboratory procedure for use of Cylcap, a product long used in testing laboratories for capping test cylinders and blocks.—Denver Fire Clay Co., 3033 Blake St., Denver 5, Colo.

ANCHOR REFRACTORY HAYDITE—Brochure describes haydite-luminite refractory concrete, discussing strength, thermal value, ease of installation, economy, and durability. Includes six tables showing results of recent tests, as well as recommendations for suitable industrial application.—Anchor Concrete Products, Inc., Refractory Haydite Division, 2450 William St., Buffalo 6, N. Y.

LIGHTWEIGHT CONCRETE INFORMATION SHEET—Revised copies of Expanded Shale Clay and Slate Institute Information Sheets No. 1, 5, and 7 are available. No. 1, "Workability is Easy," was revised March 1958; No. 5, "Suggested Design Coefficients," and No. 7, "Floor Finishing," were revised in April 1958.—Expanded Shale Clay and Slate Institute, National Press Bldg., Washington, D. C.

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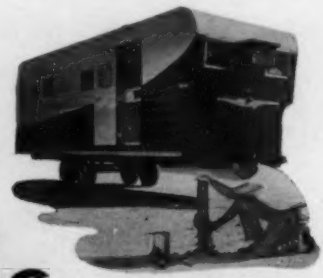

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